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Pomeroy, Johnston and Bailey, Pasadena, Calif

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FEASIBILITY STUDY ON

IN-SEWER TREATMENT METHODS

by

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Contract No. 14-12-944

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16. ABSTRACT

The possibilities of in-sewer treatments of wastewaters are broadly covered, including discussions of biological, chemical, and physical methods. The best possibilities are in the direction of biological oxidation, leading to partial reduction of the BOD, or extending to the equivalent of standard complete treatment. An appreciable reduction of BOD occurs normally in sewers. The amount of this oxidation can be estimated by equations developed in this and other closely related research. The requirements for a high degree of treatment are an adequate oxygen supply, an oxidizing culture, and time. The culture may be suspended in the form of an activated sludge, or attached to solid surfaces. The methods for supplying atmospheric or industrial oxygen are explored. The prospects for a useful degree of in-sewer treatment, especially in force mains, are very good. Under certain limited conditions the sewer can function as an efficient flocculation device in conjunction with chemical treatment.

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report discusses the possibilities of in-sewer treatments of wastewaters including discussions of biological, chemical, and physical methods. The greatest opportunities are in the direction of biological oxidation, leading to partial reduction of the BOD, or extending the equivalent of standard complete treatment. Requirements for a high degree of treatment are discussed, equations for oxidation are discussed, and methods for supplying atmospheric or industrial oxygen are explored.

Francis T. Mayo, Director
Municipal Environmental Research
Laboratory

ABSTRACT

The possibilities of in-sewer treatments of wastewaters are broadly covered, including discussions of biological, chemical, and physical methods. The greatest opportunities, by far, are in the direction of biological oxidation, leading to partial reduction of the BOD, or extending to the equivalent of standard complete treatment. An appreciable reduction of BOD occurs normally in sewers. The amount of this oxidation can be estimated by equations developed in this and other closely related research. The requirements for a high degree of treatment are an adequate oxygen supply, an oxidizing culture, and time. The culture may be suspended in the form of an activated sludge, or attached to solid surfaces. The methods for supplying atmospheric or industrial oxygen are explored. The prospects for a useful degree of in-sewer treatment, especially in force mains, are very good. Under certain limited conditions the sewer will function as an efficient flocculation device in conjunction with chemical treatment.

This report was submitted in fulfillment of Contract No. 14-12-944 by Pomeroy, Johnston, and Bailey under the sponsorship of the U. S. Environmental Protection Agency. This report covers the period from September 21, 1970, to March 1972, and work was completed as of June 1, 1971.

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SYMBOLS

Symbols used in Appendix I and Appendix II are shown in each Appendix.

C	Hazen-Williams coefficient
C_A	factor in the predictive equation to represent the increase in the exchange coefficient due to creation of additional air-water interface as result of high turbulence
C_s	solubility of oxygen, mg/l
D	pipe diameter, m or ft or in.
d	depth of flow, m or ft
d_m	mean hydraulic depth, cross section area of stream divided by surface width, m or ft
Δ, Δ_1, Δ_2	oxygen deficit and deficits at beginning and end of test reach or upstream and downstream of a waterfall, mg/l
E	rate of energy dissipation per unit mass of water, m^2/sec^3 or ft^2/sec^3, page 78
E	efficiency, mg of O_2 per kw-hr of energy in the fall, page 24

a	submerged surface transfer coefficient
f	oxygen exchange coefficient, m/hr
G	du/dx, mean velocity gradient, t^{-1}
g	acceleration of gravity, m/sec^2 or ft/sec^2
H_1, H_2	elevations of the hydraulic energy line upstream and downstream from the fall
K_2	reaeration coefficient
K_a	mural deaeration coefficient, hr^{-1}
K_H	reaeration coefficient for a waterfall, m^{-1}
$K_L a$	synonym for K_2
n	Manning coefficient
$[O_2]$	dissolved oxygen concentration
P_1	ambient pressure, atmospheres absolute
P_2	pressure of the injected air, atmospheres absolute

Q	flow, m^3/sec or cfs
R_a	rate of change of oxygen concentration due to its utilisation by the slime layer on the pipe wall
R_p	rate of supply of oxygen to a stream, mg/l-hr
R_r	rate at which oxygen is depleted by reaction in the stream
r	hydraulic radius, m
s	slope of the energy line of the stream
T	temperature, °Celsius
t	time, seconds or hours
u	velocity of flow, m/sec or fps
V	volume of air at one standard atmosphere pressure, m^3
W	Gameson weir aeration coefficient (page 17)
W	Potential work of isothermal expansion (page 42)

α	ratio of exchange coefficient in impure water to the coefficient in pure water at the same temperature and flow conditions
γ	ratio of exchange coefficient at temperature T to the coefficient at 20°C, other conditions being the same
ϕ_a	flux of oxygen from the stream to the slime layer, g/m^2-hr
ϕ_i	flux of oxygen from the atmosphere to the stream, g/m^2-hr
ν	viscosity

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This reconnaissance study was carried out primarily under Contract 14-12-944 of the Federal Environmental Protection Agency with the firm of Pomeroy, Johnston and Bailey. The project overlapped with the research on the occurrence and control of sulfide in sewerage systems, assisted by a Federal Environmental Protection Agency grant to the Los Angeles County Sanitation Districts. Studies of reaction rates of oxygen with sewage, and the data analysis for the absorption of oxygen in streams constituting essential elements of the sulfide study, consumed part of the time of Pomeroy, Johnston and Bailey assigned to this reconnaissance study on in-line treatment, and the reconnaissance study in turn benefited greatly by the painstaking data collecting job done by the Los Angeles County Sanitation Districts that made possible the development of the oxygen absorption equation, so necessary for understanding in-line oxidation.

Appreciation is expressed to John D. Parkhurst, Chief Engineer of the Los Angeles County Sanitation Districts and Project Director for the sulfide research project for making possible the cooperative aspects of the two projects. We are also greatly indebted to Charles Swanson and Gerald Stern of the Environmental Protection Agency for their advice in the planning and execution of this project.

CHAPTER 1
INTRODUCTION

The possibility of in-line treatment of sewage urges itself upon the attention of sanitary engineers. Sewage collection systems are large in both cost and size, representing investments that are typically two to four times the sums spent on treatment plants. In metropolitan systems, the sewage may spend a longer time in transit than it does in the plant. Quite naturally, the question arises as to whether the sewers may not serve a double purpose, providing treatment as well as transportation, so that the sewage will reach the end of the line partially or fully treated.

The concept is intriguing. Perhaps an overloaded treatment plant can be relieved by effecting some of the treatment in the sewers. Perhaps an integral design concept will economize on over-all construction by laying out the sewers in ways that will facilitate in-line treatment at small cost, with substantial savings in treatment plant construction. Perhaps in-sewer treatment will enhance the feasibility of withdrawing water at up-stream points for reclamation. Perhaps sulfide control methods that maximize BOD reduction should be chosen in preference to other methods of sulfide control.

The application of certain treatments to sewage in transit is not new. Principally such treatments have been for the purpose of preventing the generation of sulfide. For nearly a half century, chlorine and other chemicals have been added for sulfide control, usually having the incidental result of easing the load on the sewage treatment plant, but only by a slight amount under most conditions. The application of air to force mains, first undertaken for sulfide control by Ewald Lemcke in 1942 in Orange County, California, (1) is now widely practiced, sometimes with noticeable reductions of BOD.

Even if nothing is done toward the intentional treatment of sewage while in transit, some degree of self-purification nevertheless does

occur, because all of the oxygen absorbed at the surface of the stream is consumed in partially oxidising the sewage.

Some sewerage systems transport wastewater for distances approaching 100 km, and even longer systems may be built in the future. The potential for achieving significant in-sewer treatment during the course of the time that this wastewater is in transit should not be overlooked. A large trunk sewer of such length would probably accumulate serious concentrations of sulfide if measures were not taken to increase the oxygen supply. If sufficient oxygen were supplied to the stream, the result would be not only the prevention of sulfide buildup, but also a substantial degree of biological oxidation. The control of sulfide, partial in-sewer oxidation, and complete biological treatment in the sewer constitute a continuum. The objectives of sulfide control and of complete biological oxidation are widely separated, but the technologies overlap, and an understanding of the absorption and utilisation of oxygen in the sewer is basic for the whole range.

A major part of this report is devoted to biological oxidation in the sewer, since this process is the one that offers by far the greatest opportunity for either partial or complete in-sewer treatment of wastewater. However, the aim of the reconnaissance study is a comprehensive one, and consideration has been given to two other processes -- sedimentation and flocculation. It is found that in-sewer treatment by these processes is possible, but the opportunities for application are extremely limited.

CHAPTER 2

CONCLUSIONS

1. The most promising form of in-line sewage treatment is biological oxidation. The treatment may range from the minor BOD reduction that occurs naturally in gravity sewers to the equivalent of normal secondary treatment.

2. The three requirements for efficient biological treatment are an adequate oxidising culture, a supply of dissolved oxygen, and time.

3. If the oxygen supply is adequate, the sewage develops a capability for using oxygen at rates from 5 mg/l-hr up to as high as 20 mg/l-hr. The oxygen utilisation rate can be increased by returning activated sludge. Schemes to increase the area of slime-supporting surfaces beyond what is available on the pipe wall do not look promising.

4. Measurements were made of the rate of absorption of oxygen in sewers, and a well-substantiated predictive equation was developed. Very small flows can keep themselves well aerated, and, given a few hours of time, would accomplish good biological oxidation. In large trunks of, say, a meter or more in diameter, the rate of oxygen absorption is too small to accomplish any significant degree of oxidation.

5. Freshening the sewer atmosphere by ventilation will have no appreciable effect on the oxygen supplied to the stream by surface aeration. Enrichment of the atmosphere with oxygen has little chance of being practical.

6. The actual oxygen resources in a sewer are not limited to surface aeration of a stream, but are supplemented by the turbulence of junctions and drops. The oxygen absorption at a drop is much more than occurs with the same loss of elevation in a quietly flowing stream. Purposely designed sewage falls, U-tubes, and other devices in which the potential energy of a sewage stream is used to mix the sewage with air can be useful means for increasing in-sewer oxidation.

7. A series of low air lifts along relatively flat sewers would add to the dissolved oxygen supply and would aid in maintaining adequate sewer velocities.

8. Diffused air is a feasible way to increase in-line oxidation in large trunks.

9. Streams of screened sewage or effluents jetted at high velocity into sewage streams is a possible way to increase the oxygen supply.

10. Mechanical aerators do not appear very practical for installation in existing sewers.

11. Injection of air into force mains accomplishes a degree of biological oxidation that can be substantial if the detention time is long.

12. Refined oxygen is in a very favorable position for use in force mains or in any structure in which the sewage is placed under pressure, including U-tubes. A good degree of biological oxidation appears possible if the retention time is a few hours.

13. Refined oxygen also can be used advantageously for treatment in a trunk sewer, by injecting a supersaturated solution into the sewage stream at suitable intervals.

14. Slope-flow relationships in most sewers produce velocity gradients suitable for flocculation of aluminum hydroxide or iron hydroxide precipitates. Application of the chemicals at upstream locations where there will be flow times of 10 minutes to 30 minutes ahead of a treatment plant will result in well flocculated sewage reaching the plant. This will be advantageous only if the sewage can then be transferred to the primary settling tank without floc-disrupting turbulence.

CHAPTER 9

RECOMMENDATIONS

1. Engineers should consider the oxygen balance of sewers when they are being designed, and should choose options that favor an increased oxygen supply to the stream. Keeping the stream aerobic should be the preferred way to control sulfide and minimize other sewage odors.

2. An experimental sewer should be constructed, perhaps paralleling an existing sewer that has inadequate slope, with a series of low-head air-lift pumps and increased sewer slopes between.

3. Experiments should be made in a force main that provides a detention time of several hours, to measure the rate of oxygen usage by the slime layer, and other factors influencing the over-all biological oxidation. Air as well as oxygen should be tried. An existing force main can be found that will be suitable for these experiments.

4. A demonstration project should be undertaken in which a supercharged oxygen solution is injected into the stream in a large trunk sewer.

5. A demonstration project should be undertaken in which the stream in a large trunk sewer is aerated by diffused air.

6. A few more experiments should be made to round out the work, already started, on aeration by waterfalls and jets, using an improved apparatus. Observations should be made of the amounts of oxygen absorbed at existing points of high turbulence in sewers.

CHAPTER 4

THE OXYGEN SUPPLY FOR BIOLOGICAL OXIDATION IN SEWERS

BASIC REQUIREMENTS

There are three basic requirements for a biological purification process: 1) an oxygen supply, 2) a culture of aerobic organisms, and 3) time. The first two are largely independent and can be treated separately. The time requirement is implicit or explicit in the discussions of the first two.

This section deals with the oxygen supply. Both spontaneous dissolution of oxygen from the atmosphere and special aeration or oxygenation techniques are considered.

SCOPE OF THIS CHAPTER

There are numerous processes that can add dissolved oxygen to the wastewater in a sewerage system. These include the processes that occur spontaneously in existing systems, and that can be amplified in new systems by suitable choices of design options, and also processes or devices that are not features of conventional systems but that can be installed for the purpose of increasing the oxygen supply, either in existing or new systems.

The oxygen supplied by some of these processes may be trivial under ordinary conditions. They must nevertheless all be examined in a reconnaissance of the subject, in order that they can be evaluated. The possibilities for full secondary biological treatment in sewers are very limited, but substantial BOD reduction is often possible. Even a trivial reduction of BOD is worthwhile if it is accomplished at trivial cost.

More attention is being given to the dissolved oxygen balance in sewers as the major defense against sulfide problems. Where a process is employed to add dissolved oxygen for the purpose of sulfide control, an inevitable consequence is that there will be a reduction of the BOD or COD load reaching the treatment plant. The reduction is sometimes more than trivial.

The subject matter of this chapter will be presented under the following headings:

- --Power Efficiency as a Feature of Aeration Processes
- --Spontaneous Surface Aeration
- --Ventilation
- --Aeration at Falls and Other Points of High Turbulence
- --Air Injection Into Pressure Mains
- --U-tubes, Air Lifts
- --Installation of Aeration Devices in Sewers
- --Use of Commercial Oxygen

POWER EFFICIENCY AS A FEATURE OF AERATION PROCESSES

The choice of a process to add dissolved oxygen to wastewater will be based upon several considerations, including capital cost, operation and maintenance cost, space requirements, environmental effects, etc. It is beyond the scope of this study to discuss all of these variables for all oxygen dissolving processes. However a major item of operating cost in many of these processes is energy. The amount of oxygen dissolved per unit of energy is therefore an important characteristic, and one that merits special attention.

The energy efficiency of an oxygen-dissolving process is usually expressed as kilograms of oxygen dissolved per kilowatt hour of energy.

To be meaningful, it is necessary to know what energy is referred to. It may be the electrical energy delivered to the motor, energy trans-

mitted by the drive shaft of a motor or other prime mover, potential energy and/or kinetic energy of water, energy of adiabatic compression of air, energy of isothermal compression of air, loss of efficiency of a pump or a turbine generator, etc. There may be appropriate uses for all of these measures, but confusion often arises when the basis is not clearly stated.

The reliable evaluation of the oxygen-dissolving efficiency of an aeration process is often a difficult and tedious undertaking. It would require a longer treatise than can be included here to deal with the sources of errors in such measurements. Suffice it to say that reported efficiency should be examined critically as to methodology, and with whatever degree of skepticism is appropriate before accepting the results.

Usually the observed rate of oxygen uptake is adjusted to the rate for pure water at zero dissolved oxygen, one standard atmosphere ambient pressure, and 20 deg C, but sometimes these adjustments are not appropriate. It is unlikely that a useful purpose would be served by trying to establish a standardized efficiency rating at this time, but insofar as possible, different systems will be here compared on the customary basis.

NORMAL SURFACE AERATION

It is well established that the flux of oxygen through the surface of a stream is proportional to the oxygen deficit, that is, the difference between the dissolved oxygen concentration and the concentration that would prevail if the stream were in equilibrium with the superjacent atmosphere. The relationship is expressed by the following equation:

$$\phi_f = fD$$

where ϕ_f = oxygen flux through the stream surface, g/m^2-hr

f, defined by the equation, is designated the exchange coefficient, m/hr (f is practically the same as K_1 used by some authors)

β = oxygen deficit of the stream in respect to the superjacent atmosphere, g/m^3 or mg/l

The reaeration rate, R_f, is defined as the rate of change of oxygen concentration as a result of oxygen absorption, and is expressed in mg/l-hr. It is related to other quantities by the equation:

$$R_f = \beta f/d_m = f\beta/d_m$$

where

d_m = mean hydraulic depth, m, defined as the cross-section area of the stream divided by the surface width

The reaeration coefficient, K_2, is the proportional rate of satisfying of the oxygen deficit, expressed in reciprocal hours. It is related to other quantities by the equation:

$$K_2 = R_f/\beta = f/d_m$$

K_2 is the same as $K_1 a$ as currently used by some authors.
a is practically the same as the reciprocal of d_m.

If oxygen absorption is the only process affecting oxygen concentration, the rate of change of oxygen concentration may be represented as follows:

$$R_f = \frac{d[O_2]}{dt} = -\frac{d\beta}{dt} = K_2\beta$$

where

$[O_2]$ = oxygen concentration, mg/l

t = time, hours

Integration and insertion of limits yields the equation:

$$2.303 \log\frac{\beta_1}{\beta_2} = K_2 (t_1 - t_2)$$

(Restriction: surface aeration is the only factor influencing oxygen concentration)

It has been found (2) that the exchange coefficient for oxygen between the sewer atmosphere and the stream can be predicted quite reliably by the empirical equation:

$$F = 0.96\ \theta_A \gamma (su)^{3/8}$$

where θ_A = factor representing the creation of new interface in a highly turbulent stream

γ = temperature function, equal to unity at 20 deg C

s = slope of the energy line of the stream

u = velocity, m/sec

θ_A can be approximated as follows:

$$\theta_A = 1 + \frac{0.17\ u^2}{gd_m}$$

where g = gravitation constant

u^2/gd_m = Froude number; it is the square of the Froude number commonly used in the United States in respect to flow in streams.

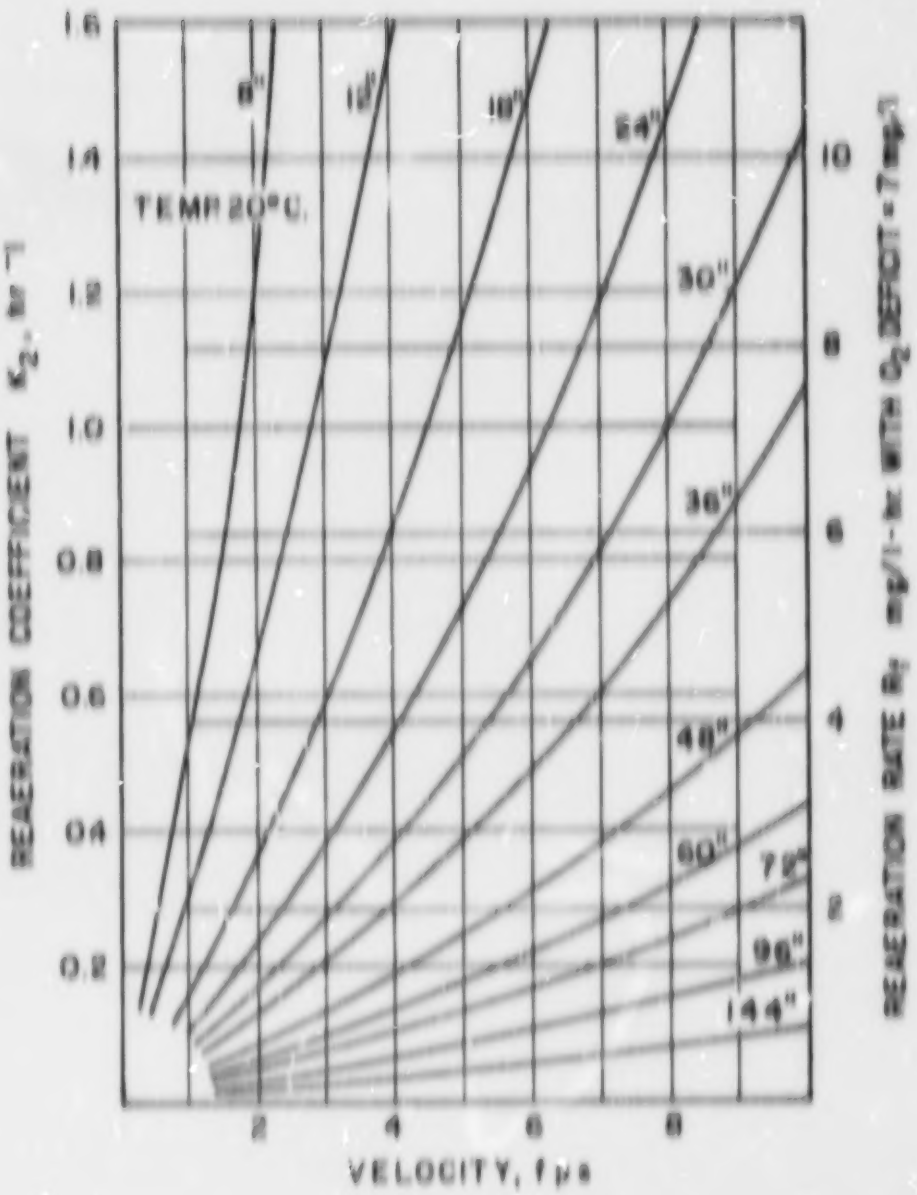
Only in quite shallow or swiftly flowing wastewater streams does θ_A exceed 1.2; mostly it is below 1.1.

The γ function of temperature is influenced by the turbulence of the stream. In very slow streams γ may increase 4 percent per degree Celsius; in turbulent streams the change is much less (3). Until such time as more information is available, the best estimate is that γ increases about 1 percent per deg C under typical flow conditions in sewers.

Figure 1 has been drawn to show values of K_2 in sewers flowing half full at 20 deg C. From the velocity and pipe diameter, slope was calculated, using a half-pipe Hazen-Williams coefficient of 109, equivalent to a full-pipe coefficient of about 124, a typical value for functioning

FIGURE 1

REAERATION RATES IN SEWERS
FLOWING HALF FULL



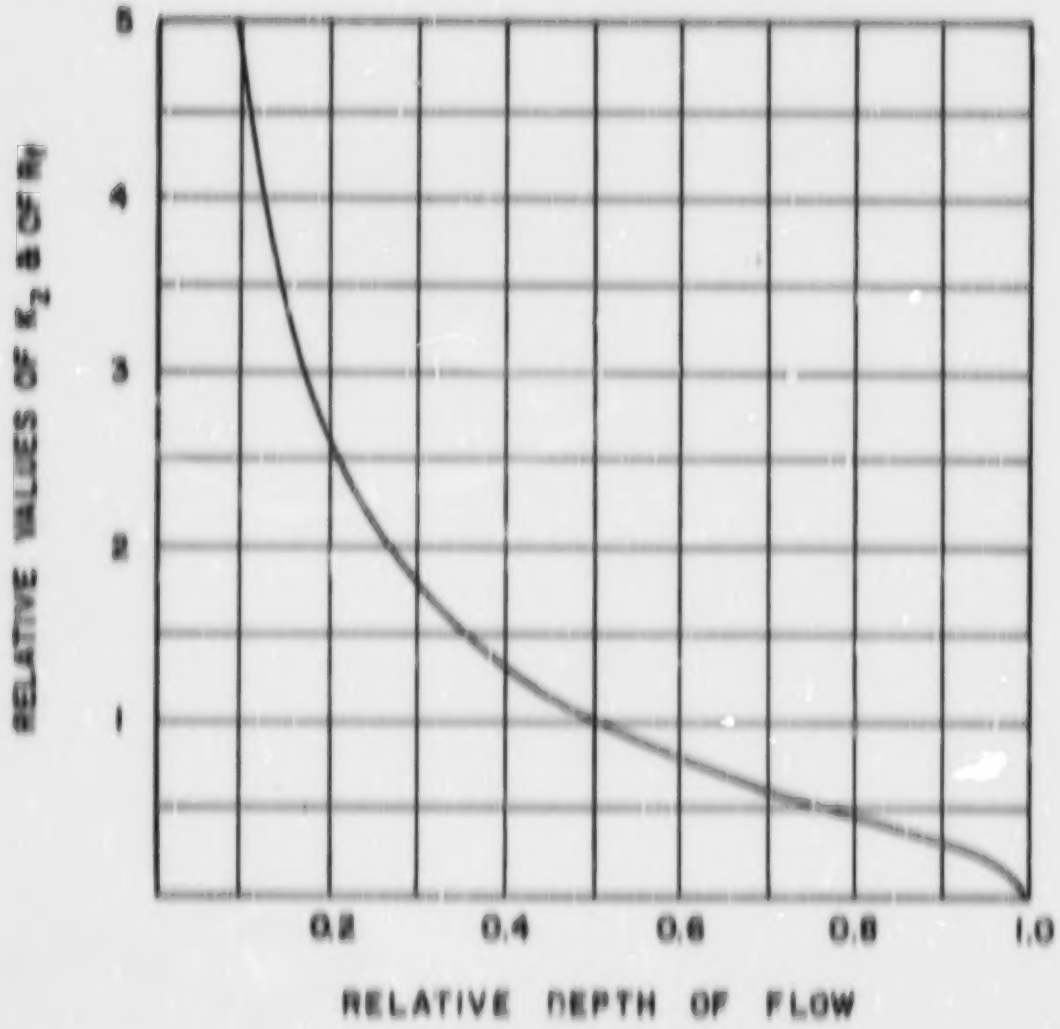
reverse. If the calculations had been made with a higher assumed Hazen-Williams coefficient, the calculated slope for the same velocity and flow would have been lower, and the calculated f and R_f would have been lower, varying inversely as the 0.7 power of the assumed value of C. For a given pipeline (fixed slope) and Q, an increase of actual C would increase the velocity and decrease the mean depth, thus increasing R_f approximately in proportion to C.

The values of f are to be multiplied by the oxygen deficit to obtain the rates of oxygen supply, R_f, in mg/l-hr. The oxygen deficit is influenced by ambient atmospheric pressure and by the oxygen content of the sewer atmosphere, which may be as low as 90 percent of normal, or less if air flow in the sewer is blocked. Assuming 95 percent as typical, and an atmospheric pressure of, say, 0.99 standard atmosphere, which would be at an elevation of about 90 m above sea level, the solubility at 20 deg C would be 8.5 mg/l. To find the rate of dissolving of oxygen when the concentration of oxygen in solution is enough to maintain good aerobic action, one should use a figure of about 7 mg/l for the deficit. The scale on the right side of graph in Figure 1 shows the oxygen supply rate under these conditions.

An increase of temperature increases f. More important, however, is the effect of temperature on the solubility of oxygen. To maintain a desired level of dissolved oxygen, a lower oxygen deficit will be available at the higher temperature.

The effects of relative depth of flow in a given sewer are as shown in Figure 2. The calculation was made for a 24-in. (0.61 m) pipe that flows at a velocity of 1.0 m/sec when half full (1.14 m/sec when full). A curve calculated for any other pipe size and velocity within reasonable limits would differ imperceptibly from Figure 2, and then only because of changes in θ_A. Factors read from Figure 2 can be used as multipliers for reaeration coefficients read from Figure 1. In using Figure 1 in this way the velocity that should be used is the velocity that

FIG. 2 RELATIVE AERATION RATES IN A SEWER AT DIFFERENT FLOWS, COMPARED TO THE RATE FOR THE HALF FILLED PIPE



would prevail if the sewer were flowing half full, not the actual velocity at the actual depth of flow.

The concept of power efficiency in the dissolution of oxygen is not very useful in respect to surface aeration of streams. The energy dissipation in a sewer where uniform flow conditions prevail is proportional to suA, in which s is the slope, u is the velocity, and A is the cross-section area of the stream. The amount of oxygen dissolved, however, is proportional to $(su)^{3/8}b$, in which b is the surface width of the stream. A wide range of power efficiencies results, depending upon slope, velocity, and size of the stream, but the efficiency is always much poorer than in processes where air is vigorously mixed with the water.

The possible significance of normal surface aeration in a sewer can be appraised by use of Figures 1 and 2. Consider a sewer 30 cm in diameter, designed for a flow velocity of 1.0 m/sec when flowing half full, but with an actual depth of 7.5 cm. The oxygen supply at low dissolved oxygen concentration would be 18 mg/l-hr. The oxygen consuming capability of wastewater and slime would soon approach this rate. If a sewer flowed under these conditions for 5 to 10 km, and then passed through a clarifier, the system would be equal to a trickling filter plant in respect to the quality of effluent. There is probably no place where this hypothesized condition exists. In large systems the wastewater is collected into larger trunks with flatter slopes and deeper flows, where the oxygen supply is too small to provide a significant degree of oxidation.

It is concluded that spontaneous aeration at the surface of the stream will seldom be sufficient to support a substantial degree of biological purification. Nevertheless, it is a significant factor in the oxygen balance.

VENTILATION

The calculation of reaeration rate, as shown by the right-side scale of

Figure 1, is valid only for an oxygen deficit of 7 mg/l. If the oxygen in the sewer atmosphere becomes partly depleted, the deficit, which is the difference between the dissolved oxygen concentration and the concentration that would be in equilibrium with the existing sewer atmosphere, is reduced. Oxygen absorption is correspondingly reduced.

The idea suggests itself that ventilation of the sewer is a way to improve conditions by eliminating this impoverishment of the sewer atmosphere. The suitability of ventilation for this purpose in any given situation can be judged by a consideration of the severity of oxygen impoverishment and the benefit that would result.

Sewer atmospheres are in fact not greatly depleted in oxygen content under normal conditions. Upstream from a complete blockage of air movement in the sewer, the oxygen concentration may sometimes be reduced to half of its normal value, but otherwise this does not happen. In small sewers the oxygen content is usually between 95 and 100 percent of normal. It is evident that ventilation would be of no significant value in such sewers. In large trunks the concentration may sometimes be as low as 90 percent of normal, or perhaps 85 percent in extreme cases. However, the oxygen dissolved by surface aeration in large trunks is a rather inconsequential amount. Even if there is a reduction of 10 to 15 percent of the normal reaeration rate because of a stale sewer atmosphere, ventilation would have a negligible effect on the oxygen balance of the stream.

AERATION AT POINTS OF HIGH TURBULENCE

In addition to surface aeration in the normally flowing streams in sewers, oxygen is added at junctions, drops, hydraulic jumps, and other places of intensive turbulence that mixes air with the water.

From the standpoint of analysing its effects, the simplest form of a high-turbulence process is a waterfall.

Falls

Measurements of aeration of water falling over river weirs were made by Gameson (4), and by Barrett, Gameson and Ogden (5). Gameson, Vandyke and Ogden (6) made measurements with an experimental fall, with the water falling into a pan 9 cm deep from heights ranging from 0.6 to 5.5 m.

Gameson showed that the effect of a weir can be modeled by an equation which may be written

$$\frac{\delta_1}{\delta_2} - 1 = W(H_1 - H_2)$$

where δ_1 and δ_2 = upstream and downstream oxygen deficits
W = weir coefficient
H_1 and H_2 = elevations of the energy line upstream and downstream from the fall

It has been shown (2) that an alternative expression is also applicable:

$$2.30 \log \frac{\delta_1}{\delta_2} = K_H(H_1 - H_2)$$

where K_H = waterfall aeration coefficient, L^{-1}

Over the range of conditions represented by the published results of Gameson and coworkers, it appeared that one equation is about as good as the other. Where the deficit ratio is infinitesimally greater than unity, $K_H = W$. Where the ratio is 2, that is, where half of the deficit remains unsatisfied, $K_H = 0.69\ W$. For the purpose of comparing efficiencies of different aeration methods, it is more convenient to use the logarithmic form.

Gameson classified the streams where tests were made as "slightly polluted," where "there appeared to be no appreciable pollution"; "moderately polluted," where the water "contained a proportion of sewage effluent"; and "grossly polluted," where the stream was largely or entirely sewage effluent. Average results were as shown in Table 1. The

averaging was done with the use of weighting factors advised by the author.

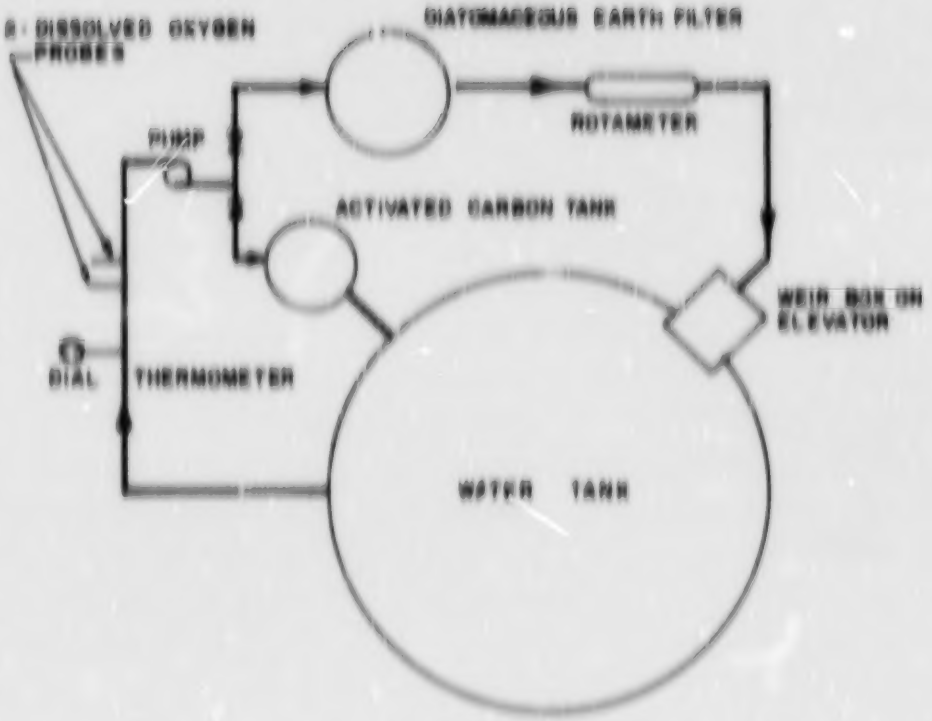
Table 1. AERATION COEFFICIENTS FROM DATA ON RIVER WEIRS

Condition	Number of locations	K_a, s^{-1}	K_H, s^{-1}
Slightly polluted	9	0.81	0.59
Moderately polluted	23	0.58	0.41
Grossly polluted	5	0.35	0.28

Experiments on aeration by waterfalls was undertaken as a part of this research project. For the purpose, the equipment shown in Figure 3 was constructed. The pump takes water from the tank and returns it by way of a diatomite filter and a rotameter to the weir box. A thermometer and duplicate oxygen probes are inserted into the pipeline from the tank. The water can also be returned to the tank by way of a carbon filter to remove impurities, but not during an oxygen absorption run. The carbon was used from time to time between runs. The weir box is positioned on an elevator, and the water comes to it by way of a hose, so the height of the fall can easily be adjusted.

After filling the tank to the desired level and then cleaning the water with the carbon filter, a small amount of cobaltous chloride and enough reagent grade sodium sulfite were added to deoxygenate the water. Recirculation over the weir was started, and as soon as the probes showed the return of oxygen, readings were started and continued at one-minute intervals. Samples were also withdrawn for analysis by the Winkler method at intervals of about 5 minutes. The experimental run was terminated after the dissolved oxygen concentration reached 6 to 7 mg/l. The logarithm of the oxygen deficit was plotted against time, and from the slope of the line the reaeration coefficient per hour was calculated.

FIGURE 3 SCHEMATIC & PHOTOGRAPH OF WATERFALL AERATION EQUIPMENT



and was corrected to 20 deg C. This correction was made on the assumption that the weir coefficient increases by 2.4 percent of the 20 deg C value for each degree rise of temperature, as indicated by the data of Gameson, Van Dyke and Ogden (6). Using the rate of energy dissipation in the fall per unit volume of water in the system, K_{μ} was determined. Table 2 shows the results.

It developed that the greatest source of error was the lack of homogeneity in the tank. This caused the results to be erratic, as may be seen by examining the Table. A better equipment design would have been one in which water is drawn from several points in the pool.

There are four measured independent variables (including width of weir) with rather scattered data. A multiple regression analysis to discern the effects of the variables would not likely be reliable, so the data were grouped and compared, as shown in Table 3. No significant differences of K_{μ} can be attributed to height of fall, depth of pool, or width of weir, but the results with flows of 152 and 151 liters per minute (35 to 40 gpm) are lower than the average for all of the lower flows by an amount 2.2 times the standard error of the difference as calculated from the standard deviations. Indications from this type of statistical examination should be viewed in the light of the limitations imposed by the basic assumptions of statistics. The reliability of experimental results is generally overestimated because the sources of error are rarely random and independent. The data in this case suggest that efficiency was less at the higher flows, but this is not a conclusive finding.

The average value of K_{μ} at 20 deg C for all runs is 0.79 m^{-1}. From the 20 deg C data of Gameson, Van Dyke and Ogden (6) for a 2-ft fall of clean water, a value of 0.82 m^{-1} is calculated for K_{μ}.

Table 2. WATERFALL AERATION EXPERIMENTS USING TAP WATER

Independent Variables			Energy of the falling water, kw	Oxygen transferred, kg/hr	K_2, s^{-1} waterfall coefficient at 20°C
Flow rate, l/min	Depth of water, cm	Height of fall, cm			
Results for 12-in. (30-cm) weir					
190	30	152	0.0331	0.0515	0.52
97	30	152	0.0142	0.0335	0.79
97	30	107	0.0099	0.0254	0.89
190	30	107	0.0231	0.0467	0.67
97	76	107	0.0099	0.0245	0.82
190	76	107	0.0231	0.0490	0.71
97	76	152	0.0142	0.0306	0.72
190	76	152	0.0331	0.0785	0.82
190	46	152	0.0378	0.0682	0.60
190	53	107	0.0231	0.0359	0.52
97	53	107	0.0099	0.0212	0.71
190	76	62	0.0132	0.0390	0.98
190	30	152	0.0378	0.0668	0.70
190	30	152	0.0331	0.0739	0.74
114	30	152	0.0285	0.0691	0.81
95	30	152	0.0236	0.0622	0.87
76	30	152	0.0189	0.0524	0.92
57	30	152	0.0142	0.0415	0.97
151	25	122	0.0301	0.0518	0.57
190	25	122	0.0265	0.0599	0.75
114	25	122	0.0227	0.0494	0.72
95	25	122	0.0189	0.0461	0.81
76	25	122	0.0151	0.0440	0.97
151	61	122	0.0301	0.0755	0.81
114	61	122	0.0227	0.0551	0.78
76	61	122	0.0151	0.0365	0.90
38	61	122	0.0076	0.0251	1.00
Results for 3-in. (7.5 cm) weir					
190	76	152	0.0331	0.0778	0.78
190	76	107	0.0231	0.0415	0.60
97	30	107	0.0099	0.0245	0.81
97	53	107	0.0099	0.0284	0.95
190	53	107	0.0231	0.0676	0.97
190	53	107	0.0231	0.0521	0.94
95	53	107	0.0165	0.0369	0.78

Table 3. EXAMINATION OF EXPERIMENTAL WATHEP/'S AERATION DATA

Effect of height of fall:			
Height range, in.	24-42	48	60
Height range, cm	61-107	122	152
Number of runs	15	12	9
Average K_H, m^{-1}	0.79	0.80	0.78
Effect of depth of pool:			
Depth range, in.	10-12	18-24	30
Depth range, cm	25-30	46-61	76
Number of runs	16	11	7
Average K_H, m^{-1}	0.785	0.81	0.775
Effect of width of weir:			
Width of weir, in.	5	12	
Width of weir, cm	7.5	30	
Number of runs	7	27	
Average K_H, m^{-1}	0.835	0.78	
Standard deviation of K_H	0.15	0.15	
Standard error of the averages	0.068	0.029	
Effect of flow:			
Flow range, gpm	10-15	20-30	35-40
Flow range, l/min	38-57	76-114	132-151
Number of runs	9	9	16
Average K_H, m^{-1}	0.83	0.83	0.74
	0.84		
Standard deviation of K_H	0.1		0.14
Standard error of the averages	0.028		0.035
Standard error of the difference	0.045		

Two tests were made with settled wastewater from the Azusa plant of the Los Angeles County Sanitation Districts, using the apparatus shown in Figure 3. (It is assumed that raw and settled wastewater would not differ significantly in oxygen absorption rate.) The sewage was retained in the tank until the dissolved oxygen concentration was low, hypochlorite was added to stop aerobic action, and then the oxygen increase was measured. Results are shown in Table 4.

Table 4. WATERFALL AERATION EXPERIMENTS USING SEWAGE
Results for 3-in. (7.5 cm) Weir

Independent variables						Energy of the falling water, kw	Oxygen transferred, kg/hr	K_H, m^{-1} waterfall coefficient at 20°C
Flow rate		Depth of water		Height of fall				
cms	l/min	in.	cm	in.	cm			
95	190	24	31	42	107	0.0251	0.0361	0.52
29	95	24	31	42	107	0.0165	0.0206	0.41

The average of these results compared to 0.79 m^{-1} for clean water would indicate α^{A}= 0.59 for settled sewage in this aeration method. Because of the erratic variations, and in view of the findings of Gameson (4) and the low α value for wastewater streams indicated by the studies of Parkhurst and Pomeroy (2), it seems prudent to use the smaller of the two results in Table 4, that is, K_H = 0.41 m^{-1}, when estimating the effect of drops in sewers. The corresponding value of α is 0.52. Using K_H = 0.41 m^{-1}, Table 5 has been calculated to show the oxygen absorption that can be expected in wastewater falls of various heights.

Table 5. EXPECTED OXYGEN ABSORPTION IN SEWAGE FALLS
Calculated with K_H = 0.41 m^{-1}

H, meters	Percent oxygen deficit satisfied	H, meters	Percent oxygen deficit satisfied
½	18	4	81
1	33	6	91.4
2	56	8	96.3
3	71	10	98.9

[A] α is the ratio of the oxygen absorption rate in an impure water to the rate in pure water, other conditions being equal.

The energy efficiency for the dissolving of oxygen in a waterfall can be calculated by the relationship

$$E = 0.367\ C_s K_H$$

where E = efficiency, kg of O_2 per kw-hr of energy in the fall
C_s = solubility of oxygen, mg/l
K_H = waterfall aeration coefficient, m^{-1}
0.367 = kg-meter equivalent ($\times 10^{-6}$) of one kw-hr

For pure water at 20 deg C, C_s = 9.02 and K_H = 0.79 m^{-1}; the efficiency is 2.62 kg/kw-hr. The efficiency of a fall for aerating wastewater for which K_H is taken to be 0.41 m^{-1} is 1.35 kg per kw-hr. Assuming that the water is pumped to produce the fall, and that the combined efficiency of motor and pump is 55 percent, about 1.4 kg of oxygen would be dissolved in pure water per kw-hr of electrical energy supplied to the motor, and about 0.75 kg per kw-hr in the case of sewage.

The assumed wire-to-water efficiency is lower than is attainable under ideal conditions, but is a reasonable estimate for average pumping installations.

As in all efficiency ratings for aeration devices, the efficiencies above stated are for zero DO, and thus would be strictly applicable only to a waterfall of very small height, starting with oxygen-free water.

The energy dissipated in a waterfall is much more efficient in promoting the dissolving of oxygen than is the energy dissipated in a smoothly flowing stream, presumably because the energy serves to mix air vigorously with the water. It follows that if part of the available elevation difference between two points along a proposed sewer can be dissipated in a fall or in a series of falls instead of being used to provide steeper slope and higher velocity, greater oxygen absorption will thereby result. An imaginary situation will illustrate this point.

It is assumed that there will be a sewer 5,000 m long with a total elevation loss of 8 m, carrying a flow of 0.20 m^3/sec (7 cfs). Three design options are considered:

1) a 91-cm (36-in.) pipe at a slope of 0.0006 to maintain a velocity of 0.61 m/sec (2 fps), with a 5-m fall at the end;
2) same slope but with five one-meter falls; and
3) a continuous slope of 0.0016.

In the last case the pipe would be 76 cm (30 in.) in diameter. Figure 4 and Table 6 illustrate the main features of the three options. Calculations of oxygen transfer to the stream follow the Table. K_2 is taken to be 0.41 m^{-1}. For purposes of simplification, it is assumed that the oxygen consumption rate equals the respective reaeration rate in all situations, and that the oxygen deficit is 7 mg/l before each fall.

FIGURE 4 HYPOTHETICAL DESIGN OPTIONS

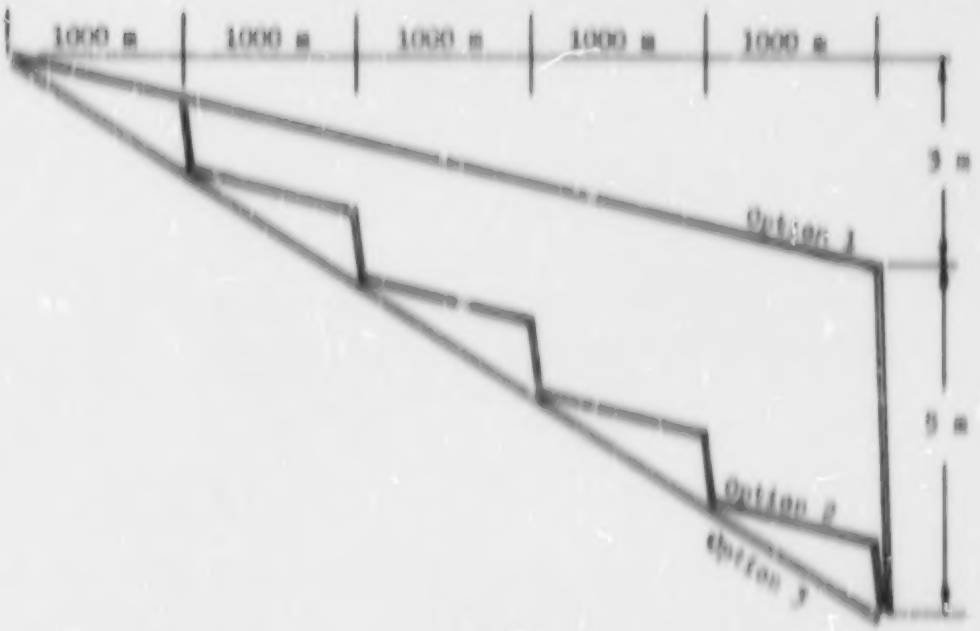


Table 6. HYDRAULIC PROPERTIES FOR THE ALTERNATIVE OPTIONS

Option	Q, m^3/min	v, m/min	D, λD,	D, cm	A/B	n	$S_{m'}$, m
1	0,20	0,61	30	91	0,9	0,0006	0,360
2	0,20	0,61	30	91	0,9	0,0006	0,360
3	0,20	0,91	30	76	0,487	0,0016	0,289

Option 1 —

The component addition of oxygen due to spontaneous surface aeration in a distance of 5,000 meters at a slope of 0.0006 can be calculated by

$$R_f = r\Delta/\Delta_m = (0.96)(1+0.17\ u^2/gd_m)(su)^{3/8}\Delta/\Delta_m$$

$$= (0.96)\left(1+0.17\frac{(0.61)^2}{0.36\ g}\right)(0.0006)(0.61)^{0.375}(7.0)(0.96)^{-1}$$

$$= 0.98 \text{ mg/l-hr}$$

Time in transit is $\frac{5000 \text{ m}}{0.61 \text{ m/sec}} = 8200 \text{ sec} = 2.28 \text{ hr}$

The amount of oxygen transferred by surface aeration is

$$(0.98 \text{ mg/l-hr})(2.28 \text{ hr}) = 2.2 \text{ mg/l}$$

The logarithmic equation for waterfall aeration for a 5-m fall, with $E_H = 0.41$ gives

$$2.303 \log\frac{\Delta_1}{\Delta_2} = 5(0.41) = 2.05$$

$$7/\Delta_2 = 7.96$$

$$\Delta_2 = 0.9 \text{ mg/l}$$

Oxygen uptake due to the fall = 7 - 0.9 = 6.1 mg/l

Total oxygen uptake = 2.2 + 6.1 = 8.3 mg/l

Option 2 --

Because the slope of the pipe for Option 2 is the same as in Option 1, the spontaneous surface aeration is also the same, and the amount of oxygen transferred is 2.2 mg/l. For each fall

$$\log \frac{\delta_1}{\delta_2} = 0.176$$

$$7/\delta_2 = 1.51$$

$$\delta_2 = 4.6$$

Oxygen uptake at each fall = 7.0 - 4.6 = 2.4 mg/l

Total oxygen uptake = 2.2 + 5 x 2.4 = 14.2 mg/l

Option 3 --

The only oxygenation occurring in this situation is that due to spontaneous surface aeration.

$$R_f = f\delta d_m^{-1} = (0.96)(1+0.17\ u^2/gd_m)(su)^{3/8}(7.0)d_m^{-1}$$

$$= (0.96)\left(1+0.17\ \frac{(0.91)^2}{0.289\ g}\right)(0.0016)(0.91)^{0.375}(7)(0.289)^{-1}$$

$$= 2.17 \text{ mg/l-hr}$$

Time in transit is $\frac{5000\ m}{0.91\ m/sec} = 5500 \text{ sec} = 1.53 \text{ hr}$

The amount of oxygen transferred in 5000 meters is

$$(2.02 \text{ mg/l-hr})(1.53 \text{ hr}) = 3.3 \text{ mg/l}$$

The results of the calculations for the three different options are summarized in Table 7. They illustrate the greater effectiveness of falls, and particularly a series of low falls in comparison with one high fall, provided that there is time for oxygen depletion between the falls. There is no conclusive evidence that a cascade made up of a series of closely spaced falls is markedly better than one large fall.

Table 7. SUMMARY OF CALCULATIONS OF OXYGEN UPTAKE
Hypothetical Design Options

Option	Q, m^3/sec	v, m/sec	n, s/m	D, cm	No. of falls	Height of fall, m	Oxygen transferred, mg/l: Surface aeration	Waterfall aeration	Total
1	0.20	0.61	0.0006	91	1	9	2.2	6.1	8.3
2	0.20	0.61	0.0006	91	9	1	2.2	9 (2.4)	14.2
3	0.20	0.91	0.0016	76	0	0	3.3		3.3

The example is imaginative, and is not intended to imply that any actual case would provide comparable alternatives, or that the amount of oxygen dissolved would be an important factor in the choice when costs and other factors are considered. It does throw some light, however, on what actually happens in sewers, and it has a bearing on the conditions of sulfide buildup in sewers, since relatively small differences of oxygen input determine whether or not sulfide problems will develop.

The larger the sewer, the lower is the rate of oxygen absorption during normal flow, as illustrated by Figure 1. A fall, however, is in general presumed to cause the same increase of oxygen concentration whether the flow is large or small. This is probably not true where the flow is so large in proportion to the drop in elevation that there is no free fall, but only an acceleration of the stream followed by a hydraulic jump. There is no doubt that oxygen absorption in such cases is greater than

in smooth flow through the same elevation loss, but it may be considerably less than calculated by the equation for waterfall aeration.

Junctions

Junctions are also usually places of high turbulence. Sometimes a tributary flow drops into a larger stream, or one or both streams are accelerated and produce a hydraulic jump or standing wave. A change of pipe diameter is also a point of abnormal turbulence.

Junctions, falls and other turbulence-producing structures are common in sewerage systems, and in the large trunks they are the principal source of dissolved oxygen. They do not, however, supply enough oxygen in the large trunks to accomplish a significant degree of biological oxidation.

In the smaller sewers both surface aeration and points of high turbulence provide oxygen in amounts that may be significant, and in hilly terrain may suffice for a substantial reduction of BOD. A notable example is the system serving the south slope of the San Pedro Hills in the Los Angeles County Sanitation Districts. It was estimated (7) that BOD reduction in the main trunk ranges 60 to 75 mg/l, in addition to an unknown amount of reduction in the small collecting sewers. This is an unplanned bonus for the hilly terrain.

The fact that this much aeration can occur in a sewerage system suggests that attention should be given, during the design of a system, to the possibility of maximizing this effect insofar as feasible alternatives may allow.

Where a pump station must be used, it may sometimes be feasible to raise the discharge elevation to allow for one or more falls downstream from the discharge point, provided proper precautions are taken against such unwanted effects as release of hydrogen sulfide and other odors. The

additional energy for pumping to a greater elevation will be approximately offset by a reduction in air requirement at the wastewater treatment plant.

Aeration in Pumps

Another example of a point of high turbulence is a centrifugal pump. Sometimes a small amount of air can be mixed with the water in a pump, using some of the energy of turbulence that is dissipated in the pump anyway, without materially reducing the efficiency of the pump. More information is needed on the dissolved oxygen-energy relationships, and on possible harmful effects on the pump. It is possible that this is an inexpensive way to add to the oxygen supply. A comparable process is the addition of air to a hydroturbine, which has been shown to yield a high efficiency of oxygen absorption in relation to the loss of efficiency of the turbine (8).

AIR INJECTION INTO PRESSURE MAINS

The injection of air into force mains and inverted siphons is a well developed technology for the prevention of sulfide buildup in such mains. A general discussion of principles, practices and results is presented in the U. S. Environmental Protection Agency Sulfide Control Manual (9). On page 5-5 of that Manual, BOD reductions of 15 mg/l, 60 mg/l, and 10 mg/l are cited for three aerated pressure mains on which data are available.

Air injection into pressure mains is not always practical, especially where the profile is irregular, or where there is very little slope so that energy dissipation by the moving air bubbles does not dissolve enough oxygen to be effective. Where air injection is practical, it is an efficient method of aeration. The energy efficiency is dependent upon several variables, including rate of air injection, rate of oxygen

utilization by the wastewater, and detention time in the main.

Whites Point Force Main of the Los Angeles County Sanitation Districts is a 61-cm (24-in.) pipe 1190-m long, with a pressure of 8.6 atmospheres (291 ft of water) at the pump station end. What the efficiency of the dissolving of oxygen would be with pure water in the pipe is not known, but measurements made under normal operating conditions at relatively low air injection rates showed that 4.36 kg of oxygen were dissolved per kw-hr of potential work of isothermal expansion of the injected air. Isothermal expansion work is here used because that is the maximum work that could be exerted by the air as it rises through the water. Actually, part of the energy is lost due to dissolving of the gas, and, if the water is moving, part of it is expended in helping to lift the water. The very good efficiency shown despite these subtractions from the energy available to produce turbulence is explained by the increase of solubility of the gases at high pressure.

It was shown, for the Whites Point Force Main, that the work of isothermal expansion of the air was 34 percent of the electrical energy applied to the motor. Thus, the efficiency was 1.48 kg of oxygen per kw-hr of electrical energy. At high injection rates the efficiency was 1.0 kg per kw-hr of electrical energy.

In three other force mains, operating at lower pressures, the efficiencies were 2.3 to 3.4 kg of O_2 per kw-hr of isothermal expansion energy. It is probable that the compressor system could be designed to convert 40 percent of the electrical energy to energy in the air, and that an oxygen-dissolving efficiency of 0.9 to 1.3 kg per kw-hr of electrical energy may be a reasonable expectation.

U-TUBES, AIR LIFTS, AND OTHER PRESSURE SYSTEMS

If diffused air were used to increase the dissolved oxygen content of a stream of sewage, it would not be economical to raise the oxygen content very high because of the consequent reduction of efficiency. If air is forced into water under high pressure, both oxygen and nitrogen can be dissolved in relatively large amounts. The injection of air into a high-pressure force main illustrates this effect. It is necessary to examine this process in some detail when considering the purposeful subjecting of the water to pressure to cause air to dissolve.

Starting with sewage containing nitrogen and argon in equilibrium with the atmosphere, but with the oxygen biologically depleted, the amounts of air that can completely dissolve are as shown in Table 8.

Table 8. AMOUNT OF AIR THAT CAN BE DISSOLVED UNDER PRESSURE IN WATER ALREADY IN EQUILIBRIUM WITH AIR AT ATMOSPHERIC PRESSURE

(Volumes of air are measured dry at one atmosphere pressure and the temperature of the water.)

Temp., deg C	Amount of air dissolved per atmosphere gage pressure	
	Volume of air dissolved, ml/l	Weight of oxygen dissolved, mg/l
15	20.0	5.68
25	19.0	5.21
35	18.2	4.87

As an illustration of the meaning of the table, suppose that it is required that 20 mg/l of oxygen be dissolved in wastewater at 25 deg C. The required pressure would be 20 ÷ 5.21 = 3.9 atm = 40 m of water. It should be noted that the table does not show the amount of gases that would dissolve if a

large excess of air were contacted by the water, because in that case the gases would not dissolve in the same ratio that they are found in air. More oxygen would be dissolved if an excess of air were diffused through the water at any given pressure.

A possible way to take advantage of the pressure effect for dissolving air is to construct a U-tube. Air would be introduced into the rising leg near the bottom of the U-tube, or would be introduced near the top of the descending leg in a sufficiently dispersed condition so that it is carried down by the water. Even if the air must be pumped in under quite high pressure, the power efficiency for getting the oxygen into solution is good if all of the air dissolves.

There is a drawback that limits the effectiveness of dissolving air under pressure. It is the reversal of the process when the water is returned to normal pressure. The tendency will be for the partial pressure of the dissolved gases to return to the ambient pressure. They will not actually reach that state by the process of bubble formation, because substantial supersaturation is required to initiate a bubble. If the water is agitated or aerated at atmospheric pressure, equilibrium will of course be approached. Without agitation, it is guessed that supersaturation of something like 40 percent would remain. This does not mean that the oxygen would be present at 140 percent of its solubility in equilibrium with the air, because the gases will not be present in the ratio found when water is equilibriated with air. The effects can be estimated.

Suffice it to say that if water has no air in it and a quantity of air is completely dissolved under a pressure of several atmospheres, or if air is used in a large excess over the amount that can dissolve, the release of pressure will leave a solution somewhat supersaturated with oxygen and nitrogen in approximately the proportions found in air-saturated water.

The benefit of the pressure would appear to be largely nullified by regassification. However, the pressure has had the useful effect of forcing the air to dissolve without a large expenditure of energy for mixing the air with the water.

Whites Point Force Main, discussed in the section on force mains, is an illustration. The normal rate of air injection is 0.5 to 0.6 m^3/min (20 ufm). Under daytime flow conditions all of this promptly dissolves, in a region where very little of the potential energy of the compressed air has been dissipated in turbulence. Most of the energy has been lost, in fact, because of the dissolving of the air. About half way up the pipeline air begins to come out of solution, but by that time a considerable amount of oxygen has been consumed biologically. The energy efficiency for the process is high as a result of this utilization of the oxygen. High efficiency would be shown in any high-pressure aeration system in which the wastewater could be held under pressure for a fairly long time. Unfortunately, there are few places where wastewater can be retained under high pressure for sufficient time for a substantial biological oxidation.

If a chamber deep in the earth were used to hold the wastewater, and there were some way to keep the sludge suspended, then activated sludge treatment could be accomplished with a very low aeration cost.

Two U-tubes were installed at the ends of force mains in Jefferson Parish, Louisiana, and four were placed in pump stations at Port Arthur, Texas. These installations were made possible by research grants from the U. S. Environmental Protection Agency (10)(11). The purpose was sulfide control, but since the U-tubes accomplish this function by dissolving oxygen, they are of interest as devices that contribute to BOD reduction in sewers.

In the existing installations, the air enters the descending leg either

by way of a venturi aspirator or by a diffuser collar. Figure 5 shows the induction of the air by an aspirator. Insofar as its effect on the dissolving of oxygen is concerned, the effect of an aspirator is to break the air up into small bubbles. It has no unique virtue not shared by other devices that use high velocity to disperse the air. Air can also be introduced by simple entrainment if there is a fall from a sufficient height into the descending leg of a U-tube. At a downward velocity of 1.5 fps, well dispersed air in amounts up to 10 percent of the volume of the water may remain dispersed; larger amounts lead to the formation of an air pocket. In one way or another the turbulence necessary to cause oxygen to dissolve is reflected in a higher hydraulic head at the U-tube inlet.

Figure 6 shows a semi-logarithmic plot of the oxygen deficits observed at various head losses through the aspirator and U-tube derived from a report on the unit by Mitchell (12). The deficits are figured on the basis of a virtual solubility of 11 mg/l. That figure represents the maximum amount of oxygen that would be retained by the water after it returns to atmospheric pressure.

The points at low head losses are quite scattered, but at the higher heads can be approximated as a straight line. The dashed line in the figure is a rough interpretation of what the trend might be for the results at lower head losses, obtained at lower air input rates.

The slope of the solid line corresponds to a waterfall aeration coefficient, K_{μ}, of 0.17 m^{-1}. For the dashed line the corresponding K_{μ} figure is 0.47 m^{-1}. Using 11.0 for C_s, the energy efficiencies at zero D.O. are 0.7 and 1.9 kg of oxygen per kw-hr. These are figures for wastewater, not clean water, and refer to the energy dissipated in the U-tube system, not the increment of electrical energy demand.

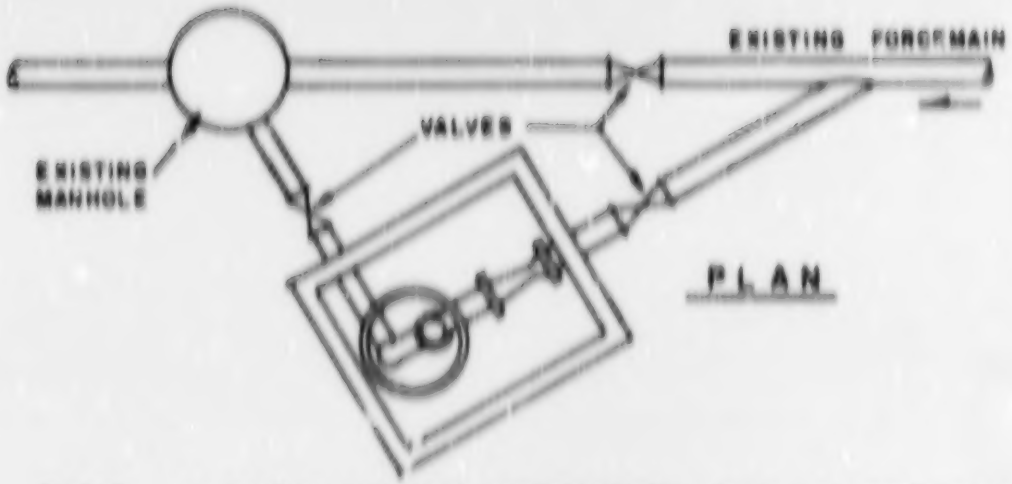


FIG. 5 U-TUBE DESIGN, JEFFERSON PARISH STATION 5

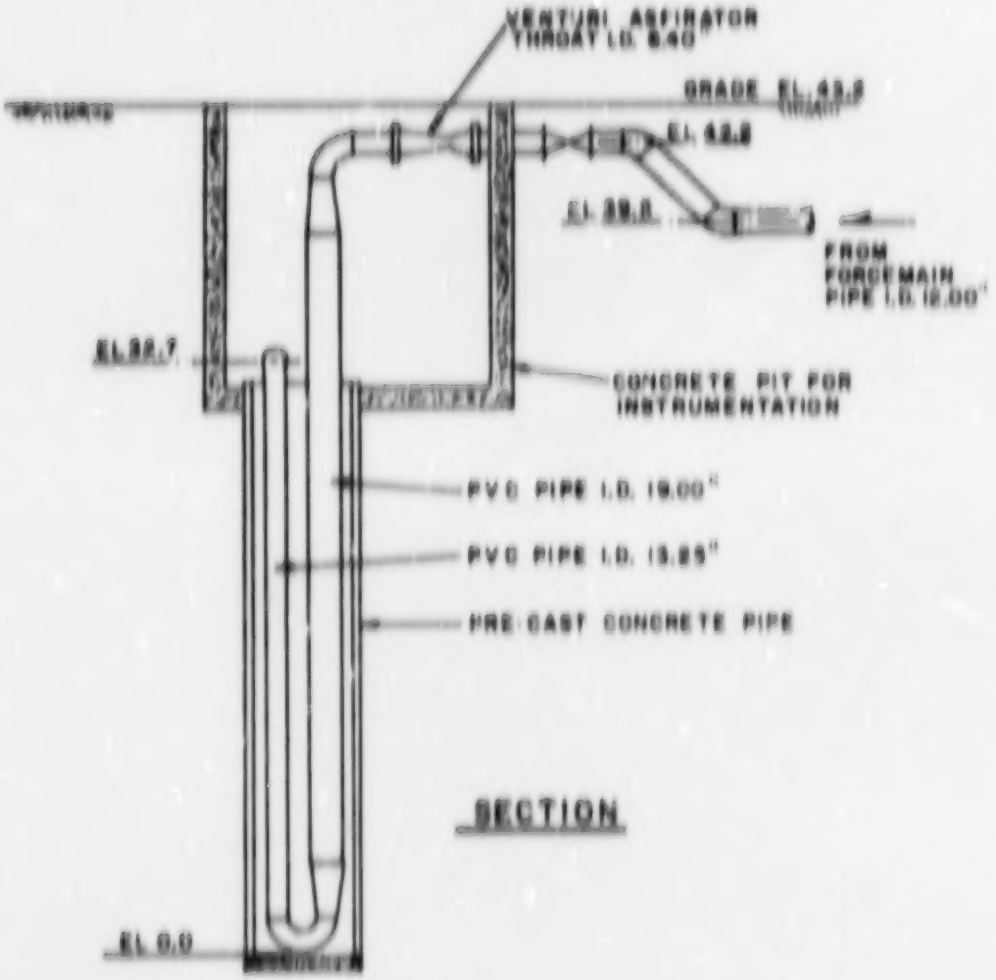
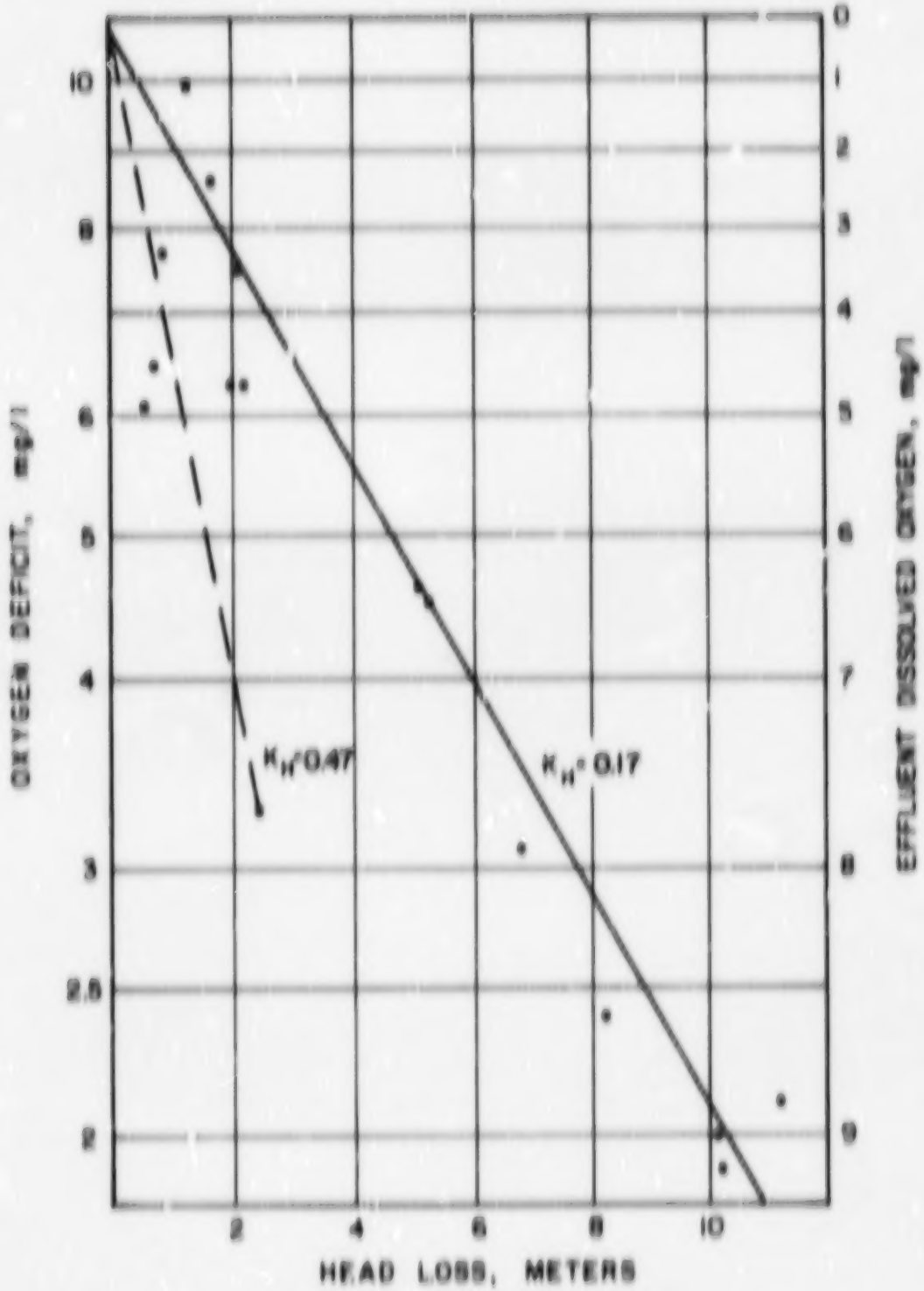


FIGURE 6, OXYGEN ABSORPTION IN A U-TUBE



More data are needed to clarify the reason for the scatter of the points, and to ascertain the efficiency of the process.

The purpose of the U-tubes in Jefferson Parish and Port Arthur is to dissolve enough oxygen to accomplish sulfide control. In this they are successful. The oxygen-dissolving efficiency is not pivotal. More important in the choice of U-tubes where they were installed was the convenience and simplicity of the installation under certain prevailing conditions. It is not likely that U-tubes would be suitable for supplying successive increments of dissolved oxygen for an extensive in-sewer BOD reduction project.

A more complete discussion of U-tube aerators is presented in the Sulfide Control Manual (9).

If air is injected into the rising leg of a U-tube, it would constitute an air lift. A zero-head air lift might be constructed alongside a sewer, having the inlet and the outlet at the same elevation. The water elevation in the descending leg would be drawn down; water would pour into it. This would be a sort of diffused air aerator with an arrangement which in effect provides greater submergence of the diffusers than if they were placed in the bottom of a sewer.

Air lifts should receive consideration for the pumping of wastewater where there is a relatively low lift. Air lifts are less efficient than centrifugal pumps. The failure of the expansion energy of the air to accomplish the equivalent amount of work in lifting the water is due mostly to the "slippage" of the air past the water, that is, the tendency of the air to rise through the water. It is precisely this dissipation of energy by air rising through the water that causes the dissolving of oxygen. If the effects of the air lift in dissolving oxygen as well as lifting the water are considered, the efficiency will look much better than when it is considered only as a pump.

The efficiency would be poorer for high lifts, exceeding, say, 10 m, because the water would be brought near to saturation with oxygen. In flat terrain, a series of low-lift stations using air lifts would prevent sulfide conditions and accomplish a significant degree of biological oxidation. The stations would be less expensive than conventional stations.

Figure 7 is a functional drawing of an air lift station. Note that the compressor takes suction from an air by-pass connecting the wet well and the downstream sewer. In this way there is no exchange of air with the outside atmosphere, and no odor release.

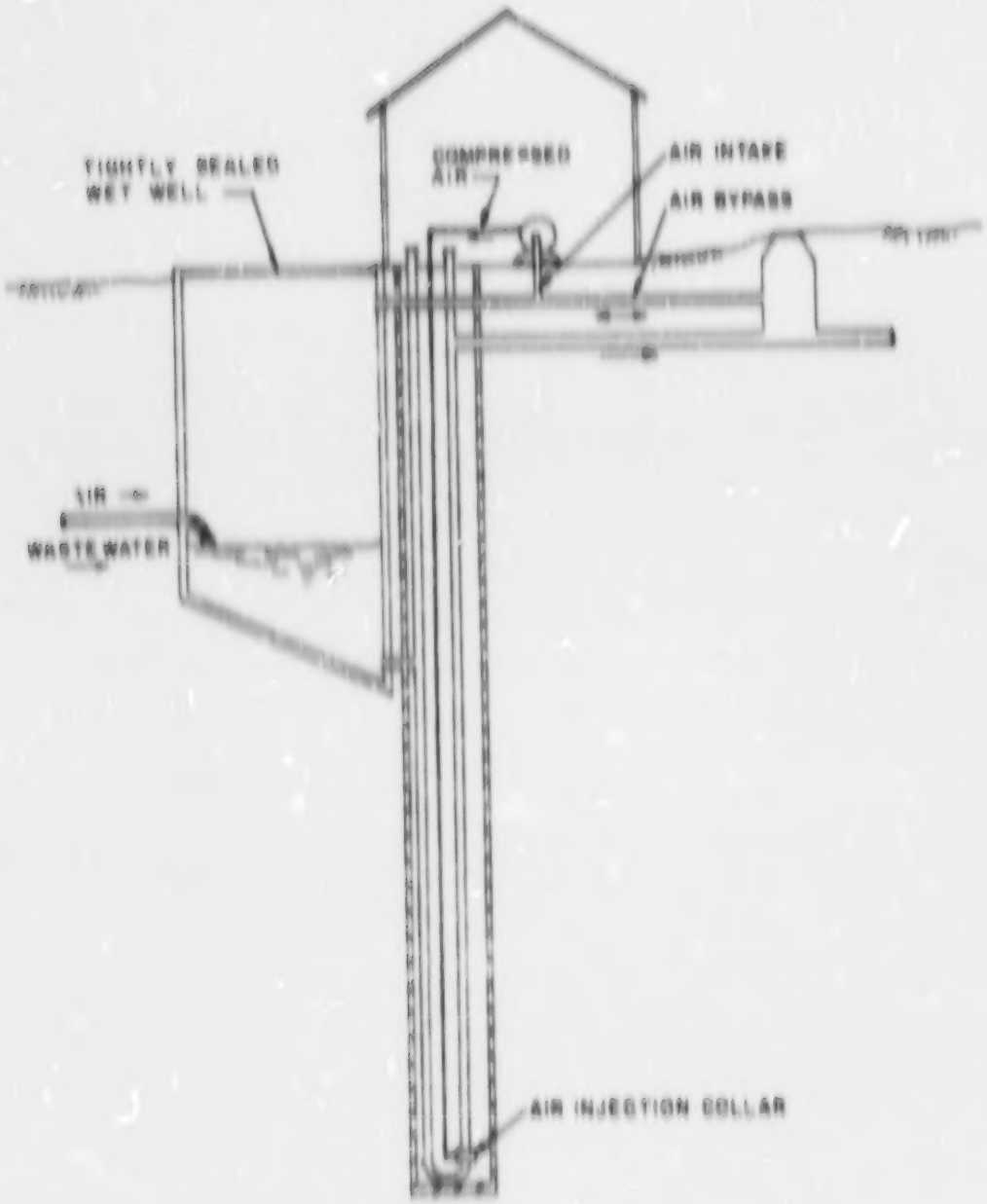
INSTALLATION OF AERATION DEVICES IN SEWERS

Many studies of methods for dissolving oxygen in water have been made over the last half century, since this is a major feature of the activated sludge process for sewage treatment. The possibility that similar methods can be used for supplying dissolved oxygen to sewage in transit must be examined.

There are basically two ways of exposing water to air for facilitating gaseous transfer. One way is to spray the water into the air, the other is to diffuse air into the water. If the objective is to bring the water close to equilibrium with the air, that is, close to saturation with oxygen, or close to zero concentration if the purpose is to strip a gas from the water, then spraying the water into the air is the most efficient process. For biological treatment it is not necessary to maintain high oxygen concentrations. Efficient processes are those in which the transfer of oxygen occurs into water with a fairly high oxygen deficit. Spraying fine droplets into the air would not be efficient from an energy standpoint, because the droplets would be brought close to saturation.

FIGURE 7

FUNCTIONAL DRAWING OF AN AIR-LIFT PUMP STATION



Aeration by Diffused Air

Aeration in activated sludge sewage treatment plants is most commonly accomplished by diffusing air into the water, using perforated pipes, ceramic plates or tubes, tubes wrapped with fabric or cord, devices that force the air out between plates or through springs, arrangements that aim to produce a high water velocity past orifices from which air is released, devices with moving parts that are intended to aid in mixing the water with the air, and equipment that channelizes the air-water mixture in a way that aims to increase or prolong the contact of the air with the water. If no external energy is applied, the energy for mixing is the potential work of isothermal expansion of the air when it is released at the bottom of the tank.

This work potential can be calculated by the equation

$$W = 0.0645\ V \log P_1/P_2$$

where W = work, kw-hr

$0.0645 = 0.02815$ (w-hr/m^3-atm) $\div \log_e 10$

V = volume of air at one standard atmosphere pressure, m^3

P_1 and P_2 = initial and final absolute pressures

Part of the energy of the air goes to circulate the water mass, which is necessary to keep sludge in suspension, but a major part of it goes to create the small-scale turbulence that leads to the dissolving of oxygen. Attempts to channelize the air-water mixture are likely to dissipate some of the energy in friction, but cannot materially increase the fraction of the energy dissipated as small-scale turbulence in the air-water mixture.

Diffusers that produce small bubbles may lead to the dissolving of 5 to 10 percent of the oxygen in the air used; large-bubble diffusers dissolve less. The fine-bubble diffusers, however, require more air pressure to

drive the air through the diffusers, part of which is due to plain friction, but part is also required for the energy expended in producing new interface against the surface tension of the water.

Several authors have treated the subject of the efficiency of aeration by diffused air. A complete review of this subject is beyond the scope of this report, but a few significant studies may be mentioned as indication of the sort of results that have been obtained.

Fauveer (13)(14) made an experimental and theoretical study of the energy efficiency of bubble aeration. The energy was calculated as the displacement energy of the free air volume against a pressure equal to the depth of submergence plus 10 cm. (For comparison with figures used elsewhere herein, the displacement work at a pressure equal to 3 m of water is 14 percent greater than the work of isothermal expansion.) He points out that in actual operation the resistance of the diffuser may be of large magnitude. The importance of small bubble size is shown, but figures that would be applicable to operating sewage treatment plants are not available.

Bewtra and Nicholas (15) studied the efficiencies of saran tube diffusers and spargers. Their curves indicate that at 20 deg C and zero dissolved oxygen the saran tubes yielded 4.0 kg of dissolved oxygen per kw-hr of energy of adiabatic compression of air at a pressure equal to the depth of submergence. At that submergence the work of isothermal compression or expansion is about 90 percent of the adiabatic compression figure. If the energy were expressed on the basis of isothermal expansion, the efficiency would be 4.2 kg of oxygen per kw-hr.

This calculation neglects the energy required to force the air through the saran tubes to produce the small bubbles. No data were given on the pressure of the air delivered to the diffusers. At normal depths, 20 percent of the energy may be expended in diffusing the air, even with clean diffusers, and a much higher proportion when the tubes have been

in service for some time. Assuming an efficiency of 90 percent in converting electrical energy into the energy of compressed air, the amount of oxygen absorbed may be 1.7 kg per kw-hr of electrical energy when the tubes are new, and some smaller amount when they are old.

For spargers the energy efficiency found by Bewtra and Nicholas is equivalent to 2.3 kg of oxygen per kw-hr of work of isothermal expansion.

These figures are for clean water. There is much uncertainty about the α factor for diffused air aeration. Sometimes $\alpha = 0.8$ is used, but the origin of this figure is not clear. Mancy and Okun (16) showed that surface active agents have no effect at low Reynolds numbers, but α may be below 0.4 at higher Reynolds numbers. It has been shown herein that α is probably between 0.4 and 0.6 for waterfall aeration. It would not be correct to assume that the same value would apply for diffused air aeration, but it seems imprudent to use a value higher than 0.6.

It is concluded that in wastewater aeration, spargers probably do not dissolve more than 0.6 kg of O_2 per kw-hr of electrical energy used, and that fine bubble diffusers when new may dissolve 1.0 kg/kw-hr. These figures are applicable only at zero D.O.

In an existing sewer, porous tubes or plates could be placed in the bottom of a manhole, or a perforated pipe or hose could be extended down the sewer. Short lengths of metal or plastic pipe could be successively coupled and pushed down the sewer, a rather tedious procedure if they would need to be removed at intervals of, say, once a year for maintenance. Holes in an iron pipe tend to enlarge by corrosion when used for diffusing air. A perforated hose or plastic pipe of sufficient flexibility to be inserted into the sewer by way of the manhole would doubtless be more practical. It would need to be weighted to prevent floating. Diffusers of this type are often used for the aeration of sewage lagoons.

Pipes or hoses running down the sewer would have the advantage of distributing the air so that oxygen would continue to be supplied while the sewage uses it. However, diffusers located in each manhole, served by a compressed air pipeline paralleling the sewer, would be able to supply enough oxygen to meet the demand in the reach to the next manhole, with only a slight sacrifice in efficiency under most conditions. It is judged that diffusers in manholes would be more practical than pipes or tubes extended down the sewer.

Considering the greater maintenance cost for small-bubble aerators, and the fact that their efficiency is likely to be less at shallow depths because of the relatively high pressure loss through the diffusers, it is concluded that perforated plastic pipes would be the preferred method if air is to be diffused into wastewater in the bottoms of manholes.

The diffused air method has the limitation that there must be enough water depth for reasonable submergence of the diffuser. With porous tubes or other fine-bubble devices in a shallow stream, the pressure loss across the diffusers is likely to represent a major part of the power requirement, with the result that power efficiency would be poorer than with deeper submergence. A larger flow of air through a diffuser with relatively large holes would not necessarily be more efficient, because some pressure loss across the orifices is necessary for distribution, else the output of each orifice will be unduly influenced by small differences in water depth or the pressure losses in the air distribution system.

The diffuser could lie longitudinally in the sewer, connected to an air supply line entering at an angle from the side and protected by a fillet of mortar so that debris would not be caught by it. Just what the shallowest depth would be for satisfactory performance is not known. It seems likely that 30 cm (1 ft) would be about the minimum. The water could be backed up somewhat by a stream-lined dam below the diffuser.

Perhaps a better arrangement would be to place the diffuser immediately

downstream from such a dam, where high velocity and turbulence would aid in the dissolving of the oxygen. The dam would be a concrete block similar in shape to a Palmer-Bowlus slab-type weir (17) but with a blunt downstream face, or an undercut face, with the diffuser in a crosswise position. Figure 8 shows ways that this could be done.

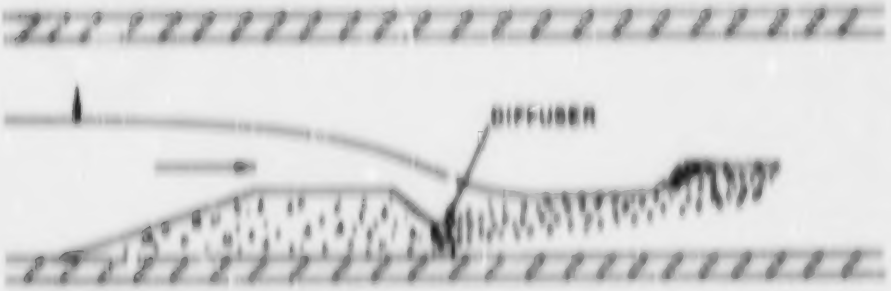
Surface Aerators

Rather than to blow air through the water, a dispersion of fine bubbles can be produced by devices that beat the surface of the water. Kessener (18) developed the brush aerator in the 1920's. It is a device turning on a horizontal axis, with metal projections dipping into the water. Pasveer (19) made careful measurements of efficiency, and optimized physical features of the installation, finally attaining an efficiency of 2.5 kg of O_2 per net kw-hr. This was at zero D.O. and 10 deg C. On the basis of assumptions made by Pasveer, the efficiency at 20 deg C would be 2.95 kg per net kw-hr, or 2.44 kg/kw-hr of applied electrical energy. With less than optimum efficiency of motor and speed reducer, the efficiency might be as low as 2.0 kg/kw-hr. In wastewater it might be 1.0 to 1.2 kg of oxygen per kw-hr of electrical energy.

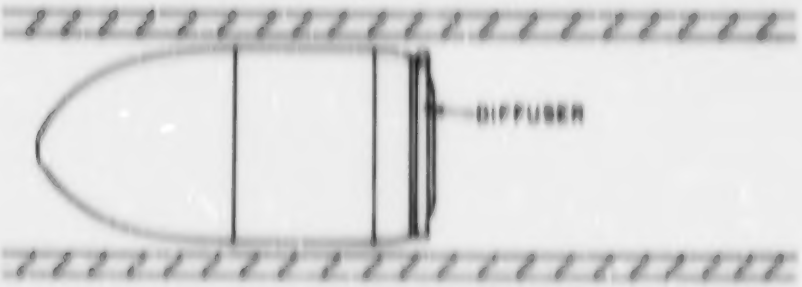
The greatest use of brush aeration in recent years in the United States has been in "racetrack" sewage treatment units, in which the aerator serves the additional function of moving the water around the racetrack channel.

Another type of surface aerator now much used in the United States is a device rotating on a vertical shaft, with blades that stir the water and throw it outward as a heavy spray. For activated sludge plants, the designs take into account the need to expend part of the power to keep the aeration basin stirred. A draft tube under the aerator often serves this purpose. In a sewer, the device would need only to stir the surface.

FIGURE 8
POSSIBLE WAYS TO DIFFUSE AIR INTO A SEWAGE STREAM

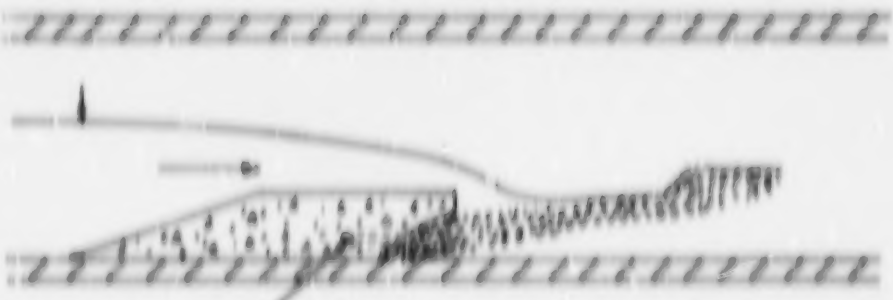


SECTION



PLAN

OPTION - A -



SECTION

OPTION - B -

The installation of mechanical aerators in an existing sewer would be difficult under most circumstances. Horizontal-shaft aerators would face the severe handicaps of limited space and of maintenance problems of the moving parts submerged in sewage. Vertical-shaft as well as horizontal-shaft aerators would need to be designed with consideration for varying flow levels. A streamlined weir could be used to reduce the rise and fall of the water, and might under some circumstances serve to stabilise the water elevation sufficiently for reasonably efficient operation. There is the possibility of using pontoons to float the aerator, or perhaps to float the rotor, with a telescoping shaft to a fixed motor.

The sort of special structures that would be needed for installation of conventional mechanical aeration devices are likely to be uneconomical because of the limited amount of oxygen that each such device could add to the stream. Sufficient oxidation to justify the cost of a special structure would require the impounding of the sewage for a time, in which case the facility would be classed as an upstream plant rather than in-line treatment.

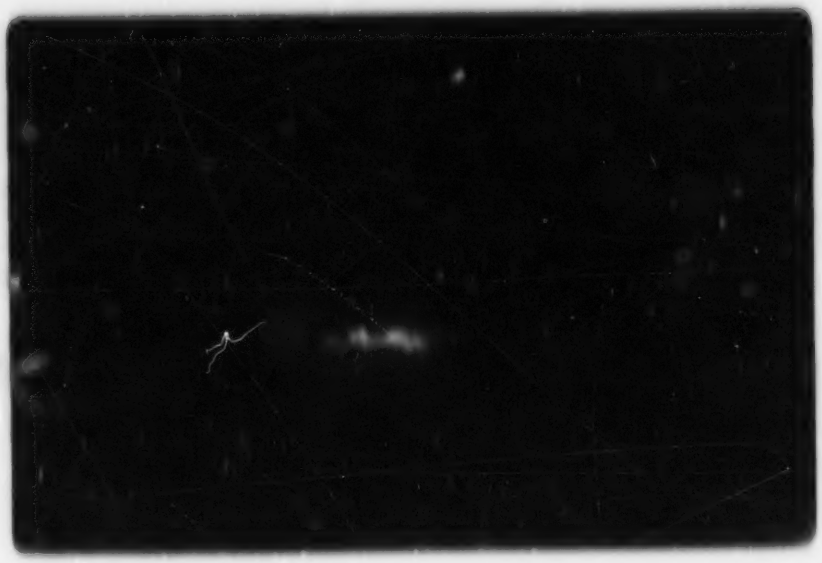
Jet Aeration

Another method for mixing air with water that could be used in a sewer is jet aeration. A stream of water under pressure is jetted by way of a nozzle into a larger stream or body of water, whereby air is entrained and broken up into fine bubbles. Only a plain nozzle is here considered.

Figure 9 illustrates a jet aeration installation serving to treat an industrial wastewater. Water from the pond is pumped into the header, which delivers it to sixteen nozzles injecting it back into the pond. The nozzles are 2.5 cm in diameter, operating under a head of about 18 m. At the time that the photograph was taken, the water level was below the design level. This causes rather too much fraying of the jet before it reaches the water surface; however, it should be moderately frayed for

FIGURE 9

A JET AERATION INSTALLATION



maximum entrainment of air that is then driven into the pond. Around the jet the water appears milky from the fine air bubbles so produced.

A few tests of jet aeration were made with the apparatus shown in Figure 3. Results are given in Table 9.

Table 9. EXPERIMENTS WITH NOZZLE AERATION

Independent variables					Energy		Oxygen
Flow rate, l/min	Depth of water, cm	Height of fall, cm	Nozzle dia, cm	Pressure gauge, psig	of the falling water, kw	Oxygen transferred, kg/hr	transfer efficiency, kg/kw-hr
Results using clean water							
42	76	10	1.12	4.5	0.025	0.0257	1.03
42	76	20	1.12	4.5	0.0258	0.0360	1.40
55	76	20	1.12	7.25	0.0489	0.0617	1.26
102	76	20	1.91	2.90	0.046	0.0649	1.41
12	76	20	0.46	9.4	0.0142	0.0269	1.90
22	61	20	0.46	21.25	0.0542	0.0291	0.54
Results using sewage							
42	76	20	1.12	5	0.0269	0.0245	0.90
55	76	20	1.12	7.8	0.0531	0.0550	1.04

The average efficiency in clean water was 1.28 kg of oxygen per kw-hr of potential plus kinetic energy in the jet stream, or somewhat less than half the value found for waterfalls. It is possible that better results would be obtained under optimised conditions. The best result found was 1.9 kg per kw-hr.

Even if the energy efficiency is not high, the process may sometimes be useful because of the relatively simple installation, especially if water under pressure is available, as at a pump station. If used for in-sewer aeration, a supply of water under high pressure would be carried by a

pipe paralleling the sewer, with branches leading to nozzles at suitable spacings, not necessarily at manholes. If wastewater from the sewers is used for the jets, it would be necessary for it to be screened to avoid clogging of the nozzles.

Venturi Aspirators

The high shear in the throat of a venturi that is aspirating air causes the air to be broken up into small bubbles. The device has no unique oxygen-dissolving potential not accounted for by the high velocity, but sometimes it has the convenient characteristic of being able to function also as a pump. It has been used for the purpose of dispersing air in flotation separators, and its application ahead of a U-tube is illustrated in Figure 5.

A venturi could be submerged in the sewage stream with a small pipe to bring air to it from above the water surface. Probably a better plan would be to locate the venturi above the water, perhaps at a considerable distance, with the air-in-water dispersion carried by pipeline into the wastewater stream. Screened wastewater could be pumped to operate the device.

The energy efficiency of a venturi as an oxygen-dissolving device is not known.

Comparisons and Practical Installations for In-Sewer Aeration

As already indicated, surface aerators do not look promising as devices to install in sewers. Since only a few mg/l of oxygen could be added to the wastewater stream at one time it would be necessary to install many of them to obtain a substantial effect. Each installation would be quite expensive.

Aeration by simple jets is a process that deserves further investigation. An experimental installation should be made to secure the necessary information. Judgment about the applicability of the process for use in severe needs to be deferred until more is known about the operating characteristics.

Unless venturis show some advantages not presently recognised, they appear unlikely to be favourably considered.

It appears from these considerations that if an aeration facility were to be installed in an existing sewer it probably would be of the diffused air type, using a perforated plastic pipe for the diffuser.

THE USE OF REFINED OXYGEN

Principles of Oxygen Application

The methods considered thus far for augmenting the dissolved oxygen supply of a sewage stream depend upon contacting the wastewater with air, whereby oxygen is extracted from the air by the oxygen-deficient water. The use of refined oxygen is another approach to the same objectives, but it involves technologies which in some respects are quite different, and in some cases it permits the attainment of results not possible by the use of air.

The term "refined oxygen" is here used to mean oxygen extracted from air either by liquefaction and fractional distillation or by the molecular sieve method. The distillation method is used where the quantities produced are greater than about 25 to 30 tons per day. The purity is generally about 98 percent. It is generally produced as a gas and used near the site of the refining facility. Alternatively it may be delivered as a liquid. It is much more expensive in this form. The molecular sieve method does not require liquefaction. This is the more practical method where the amounts produced are too small for the economical installation and operation of liquefaction equipment. The purity of the oxygen produced by this method is generally 85 to 90 percent. It is either used at the site where it is produced or delivered by pipeline in the form of a gas.

The relative costs of dissolved oxygen from air and from refined oxygen vary widely. The principal cost for aeration is power. Considering basic energy efficiencies of aeration processes, mechanical and electrical losses, the α factor, and the fact that the oxygen dissolves into water containing some dissolved oxygen rather than a solution devoid of oxygen, it seems probable that most functioning systems are dissolving from 0.5 to 1.0 kg of oxygen per kw-hr of electrical energy purchased. Prices of electrical energy vary with location and demand, so the cost of electric power per kg of dissolved oxygen is quite variable. Capital costs are relatively high, and operating costs other than power are a significant factor.

The prices for refined oxygen, too, vary over a wide range, depending on location and demand as well as electric power costs. For quite large demand rates the cost is minimized by the installation of oxygen-producing plants. The equipment and maintenance costs for in-sewer application of oxygen are lower than for attaining the same results by aeration.

In most situations the over-all cost for using refined oxygen is higher than for aeration. Oxygen has advantages, however, that in some situations may outweigh cost. This is especially true when the use is for in-sewer treatment. The nitrogen in air interferes with the dissolving of really large amounts of oxygen. When refined oxygen is used, much higher concentrations can be attained. A point application of oxygen in a sewerage system may suffice to keep the stream aerobic for one to five miles downstream.

The dissolved nitrogen content of sewage must be taken into account, however, even when pure oxygen is used. It can be assumed that sewage will usually contain dissolved nitrogen in an amount approximately in equilibrium with the atmosphere, which means that its vapor pressure will be about 0.79 atmospheres. If pure oxygen were added only in the amount that would completely dissolve at equilibrium at atmospheric pressure, the concentration would be only 8 to 9 mg/l at ordinary sewage temperatures, that is, it would be the same as for water in equilibrium with the air. If a larger amount is added, some will remain undissolved, and some nitrogen will come out of solution, so that residual gas will be a mixture of oxygen and nitrogen. If higher concentrations in solution are desired, nitrogen-diluted oxygen must be vented, in amounts increasing as the dissolved oxygen concentration is increased. Even though the solubility of oxygen is about 40 mg/l, this concentration cannot be approached in a one-step operation without wasting a substantial part of the oxygen. For example, to obtain a concentration of 30 mg/l at 25°C would require the wasting of 29 percent of the applied oxygen.

Appendix II presents equations for calculating the amounts of oxygen that must be applied in a one-step operation to obtain various concentrations. Figure 10 shows results of the calculations for 25°C and pressures of 1.0, 1.3 and 1.6 atmospheres. The higher two pressures correspond to submergences in about 3 and 6 meters of water.

In an actual case the oxygen will be somewhat less than 100 percent purity, and the amount dissolved will accordingly be less.

If the oxygen is in contact with the liquid for a prolonged time and oxygen is consumed biologically during that period, as in an activated sludge plant using oxygen, the efficiency is of course much better.

High oxygen concentrations may be reached by dissolving oxygen under higher pressures. Table 10 shows the solubility of pure oxygen at various temperatures and pressures.

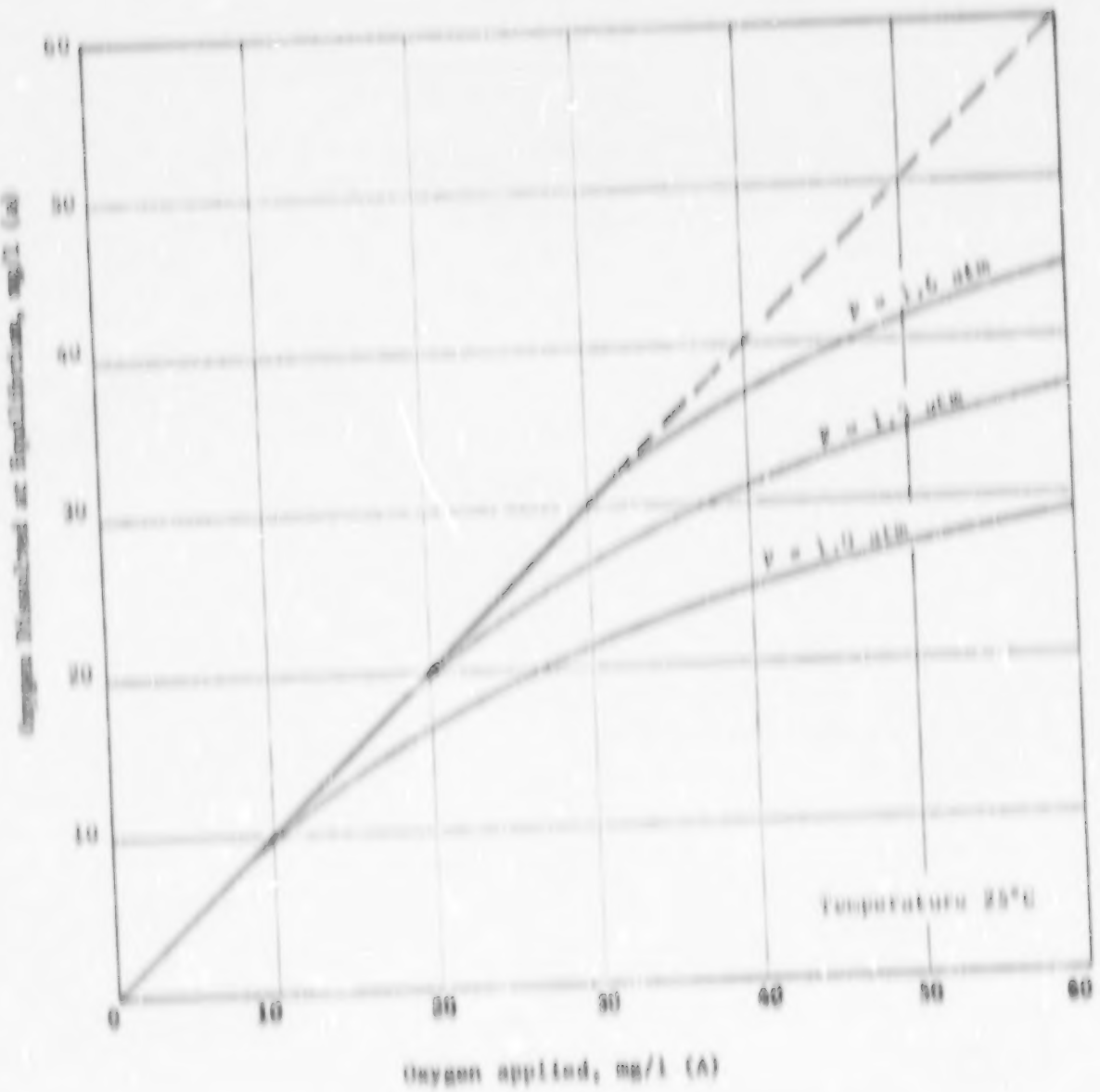
Table 10. SOLUBILITY OF OXYGEN IN PURE WATER, mg/l

Temp, deg C	1 atm	2 atm	5 atm	10 atm	20 atm
10	54	107	268	537	1074
15	48	96	240	480	960
20	43	87	217	434	868
25	39	79	197	393	786
30	36	72	179	359	718
35	33	66	166	332	663

In water that contains the normal amount of nitrogen, the solubility of oxygen will be reduced, but this effect is negligible at high pressures. The impurities in sewage, other than nitrogen, do not have any significant effect on the solubility, although they affect the rate of dissolving.

FIGURE 10

CALCULATIONS OF ABSORPTION OF PURE OXYGEN
IN WASTEWATER DEVOID OF OXYGEN



If water that has been charged with oxygen at high pressure is then returned to atmospheric pressure, the water will be supersaturated. If the supersaturation is great enough, there will be effervescence, that is, spontaneous formation of bubbles. The tendency to effervesce is increased by turbulence. There is not much practical information about oxygen losses by effervescence, but from such evidence as is available it seems likely that where the vapor pressures of the gases total 1.5 atmospheres, there will not be excessive effervescence. This would be 0.8 atmosphere of nitrogen and 0.7 atmosphere of oxygen, corresponding to about 30 mg/l of dissolved oxygen. There probably will not be very much loss even at 40 mg/l of dissolved oxygen unless there is considerable turbulence. There will, of course, be loss by transfer to the atmosphere across the phase boundary. The same equation that serves to calculate oxygen absorption when there is an oxygen deficit can be applied to the transfer in the opposite direction when there is an oxygen surplus. The loss by this mechanism will be quite small in large trunks.

Techniques for Using Oxygen in Sewers

One idea for using oxygen is to enrich the sewer atmosphere, so as to increase surface oxygen absorption. It would be difficult to avoid loss of part of the enriched atmosphere to the outside or by ingress of outside air or the downstream air draft. It would be particularly difficult to avoid losses from small sewers with lateral connections, and in large trunks the rate of aeration at the surface of the stream is so small that there would not be very much benefit even from an atmosphere of pure oxygen.

Another possibility is to follow the example of Unox sewage treatment plants. The oxygen could be bubbled through the stream in the bottom of a manhole, and then the enriched atmosphere could be used to aerate the water at the next manhole downstream, and so on down the sewer until most of the oxygen is used. This might be a feasible method under favorable circumstances, but in most situations it would be difficult to avoid unacceptable oxygen losses.

Oxygen Injection into Pressure Mains

This is a method already in extensive use in the United Kingdom. A report on one such installation at Stevenage was presented by Boon and Lister (paper presented Sept. 13, 1974, at Seventh Conference of the International Association of Water Pollution Research at Paris).

When air is injected into a force main, it is usually applied essentially continuously, because the water in the main must be supplied with dissolved oxygen by a near-continuous stream of air bubbles moving up the pipe, but when oxygen is used it can generally be dissolved completely in sufficient amount to last for the length of time that the water spends in the pipe. In the Stevenage installation the oxygen is injected into the suction of the pump while the pump is running, so that all of the wastewater receives the same oxygen dosage.

There no doubt will be places where not enough oxygen can be supplied in this way. In these cases a stream of oxygen may need to be allowed to bubble up the pipe, or it may be injected at more than one point.

The dosage of oxygen applied in the Stevenage case is 75 mg/l. Of this amount, an average of 59 mg/l react, leaving 16 mg/l in solution at the end of the main. At the prevailing temperature of the wastewater, 15°C, this much oxygen has a partial pressure of 0.33 atm. With 0.79 atm vapor pressure of nitrogen, this gives a total of 1.12 atm, which is not enough to cause appreciable loss. At times, however, the concentration reaches 36 mg/l. Some may be lost at this concentration, since the sum of the partial pressures is 1.46 atm.

Very long pressure mains, with retention times of many hours, afford the possibility of a rather high degree of biological oxidation. It would be difficult in such cases to supply the oxygen requirement by air injection, but it could be done easily by repeated injections of pure oxygen. Consideration may need to be given to the possible effects of low pH due to retention of CO_2, but it seems unlikely that there would be any serious problem on this account.

Oxygen does not automatically go into solution just because it is held under pressure in contact with the water. Mixing energy is required. The slope of the pipe is therefore a critical factor. Oxygen injection into force mains with little slope has, in two cases, failed even to provide sulfide control.

Oxygen injection into U-tubes

In Port Arthur, Texas, oxygen is injected into two U-tubes. They are only 3 m deep, and part of the oxygen escapes unused. The method should be very efficient in a deep U-tube, where the pressure will force the oxygen into solution rapidly.

The required depth to dissolve any given amount of oxygen cannot be stated at this time. Experimental studies are needed. Certainly the pressure needs to be greater than the equilibrium pressure for the amount of oxygen to be dissolved, and if the objective is to dissolve the amount of oxygen that can be carried without loss by effervescence at the U-tube discharge, the equilibrium pressure would be only 1.3 to 1.4 atm absolute, which would be at a depth of 3 m to 4 m. However, the U-tube will need to be considerably deeper to assure the complete dissolving of the applied oxygen.

Release of a Supersaturated Oxygen Solution into the Stream

Oxygen can be dissolved readily at high pressure to produce a solution containing oxygen at concentrations up to several hundred mg/l. Oxygen

would start to come out of solution upon any reduction of pressure. However, if pressure reduction occurs only at a nozzle submerged in a body of water, the jetted stream draws a large volume of water into itself, diluting the oxygen concentration until the mixture is no longer supersaturated. If any bubbles form, it is only momentarily, and they are so minute that they quickly dissolve.

Figure 11 is a photograph of an installation of this type constructed in 1970 to provide an oxygen supplement for an overloaded activated sludge plant. The tank for liquid oxygen is in the background. In the foreground is the oxygen dissolving tank, shown in greater detail in Figure 12. Oxygen is released under pressure into the tank, forming a gas pocket in the top. Wastewater is pumped into the top of the tank, jetting downward so as to dissolve oxygen. The water level seeks an equilibrium point where the oxygen is dissolved as fast as it is fed. Any amount of oxygen can be fed, up to a maximum dissolving rate fixed by the pressure and the water flow. The high-pressure solution is conveyed to the bottom of the aeration tank, where it is released by way of a nozzle. No release of oxygen bubbles is observed in the area where the oxygen solution is injected. The nozzle and pump must be matched for efficient operation in an installation of this type.

A similar system could be applied for the supplying of oxygen to the stream in the sewer. Screened sewage or any other available water could be used for preparing the oxygen solution. Figure 13 shows how the process might be applied in a sewer. A pressure pipeline might carry the oxygen solution to a number of points of application. If 20 to 30 mg/l of oxygen are supplied at each point, it would not require very many such applications to support a substantial degree of in-sewer BOD reduction.

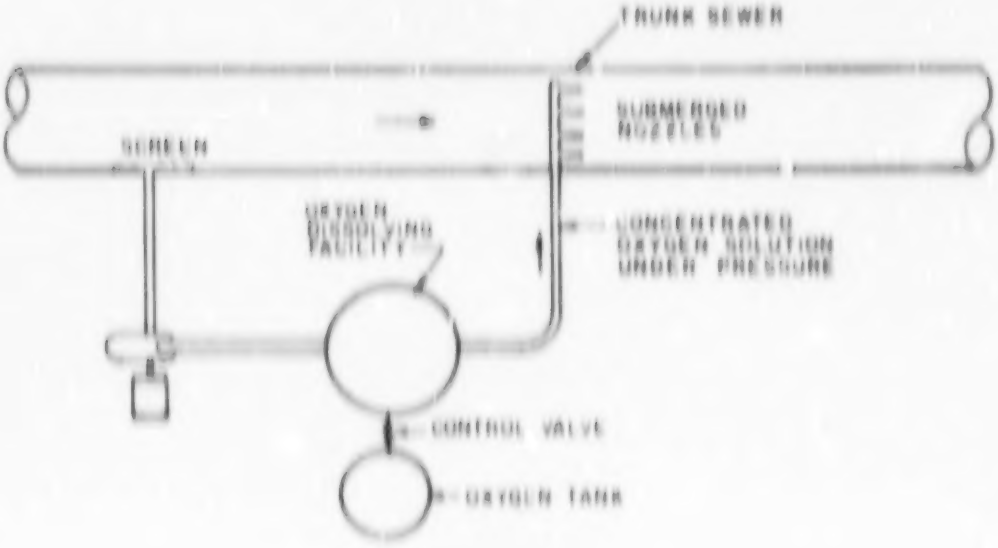
FIGURE 11. INSTALLATION TO PROVIDE OXYGEN SUPPLEMENT FOR OVERLOADED ACTIVATED SLUDGE PLANT

FIGURE 12.

GREATER DETAIL OF OXYGEN-DISSOLVING TANK

FIGURE 13.

TREATING THE STREAM IN A SEWER WITH OXYGEN



CHAPTER 5

THE OXIDISING CULTURE FOR BIOLOGICAL OXIDATION IN SEWERS

An ample supply of dissolved oxygen, which can be assured by methods described in Chapter 4, will not result in biological purification if there is not an adequate culture of aerobic microbes. Neither will a culture of microbes accomplish the desired result if they are not supplied with oxygen. This chapter discusses the ways that an oxidising culture can be provided and indicates the sort of information that is needed for estimating the oxygen requirement.

A biological culture capable of purifying sewage may exist either as a suspension of microbes in the wastewater or as a biological slime on some supporting medium. The two ways in which the oxidising culture can exist are the bases of the two principal types of secondary sewage treatment plants, that is, activated sludge and trickling filters. Both suspended and attached cultures may be involved in a single system of in-sewer biological treatment, but in most applications one type or the other will predominate.

THE INDIGENOUS BIOLOGICAL FLOC

A report on "Self-purification in Sewers" (Appendix I of this report) encompasses most of what is known about aerobic processes in sewers. The experimental and deductive work for that paper was done largely as a part of the preparation of this report, and, in a lesser degree, as an element of the study of sulfide control in sewers, also funded by the U. S. Environmental Protection Agency. It was shown therein that sewage near the source does not have a high degree of biological activity, as measured by the rate at which it can use oxygen. By the time wastewater from residences reaches the public sewer, it is seeded with a great array of microbes, but seeding alone is not sufficient. For an effective self-

of biochemical change, there must be a high population density of species adapted to that environment.

If sewage is kept supplied with oxygen, suitable species will proliferate and biological oxidation will accelerate. This change is illustrated in Curves A and B of Figure 6 of Appendix I. To obtain a curve such as those illustrated, a sample of sewage is kept aerated in a bucket, and from time to time a portion is taken out, tested for oxygen reaction rate, and then returned to the bucket. The testing is done by holding the sub-sample in a vessel where it is stirred out of contact with the air, and oxygen concentration is monitored by use of an oxygen probe. Each point in Figure 6 of Appendix I is the result of one such oxygen reaction rate determination. The results are expressed as milligrams per liter-hour.

The culture obtained by aerating the sewage in a bucket is a non-flocculent, non-settling suspension.

In a sewer, the oxygen reaction rate increases more rapidly than it does when the sewage is aerated in a bucket. Generally the suspended solids increase, the increase being in the flocculent, settleable portion. These changes reflect the sloughing of biologically active slimes from the pipe wall, providing a material resembling activated sludge.

If the stream of sewage remains fairly well aerated, quite high reaction rates are reached. The most notable example of this is in the South Slope Trunk of the Los Angeles County Sanitation Districts. The trunk receives well aerated sewage from tributary sewers coming off the San Pedro Hills. The trunk is not particularly well aerated; in fact, some sulfide is produced, but a sufficient degree of biological activity is maintained so that the flow reaches the Whites Point pump station, after an average transit time of three hours in the trunk, with an oxygen reaction rate as high as 15 mg/l-hr in the winter when the sewage temperature averages 19 deg C, and rises to between 20 and 25 mg/l-hr in the summer when the sewage

temperature is 26 to 27 deg C. The sewage then spends about 50 minutes in the aerated Whites Point force main, where it uses about 16 mg/l of oxygen. Altogether, it was estimated (Appendix I) that from the time of its entrance into the South Slope trunk until its discharge from the force main, the sewage has used 60 to 75 mg/l of oxygen. This is in addition to substantial oxidation presumably occurring in the steep tributary lines.

Curves D, E and F were made with sewages from major trunks tributary to the Los Angeles City Hyperion Plant and the Joint Water Pollution Control Plant of the Los Angeles County Sanitation Districts. These sewages had spent about ten hours in transit. A comparison of their behavior with that of fresh samples shows that there were major differences in their biological condition. Samples taken near the source and aerated in the laboratory never reached a condition resembling the sewage from the large trunks. Furthermore, a sample of fresh sewage held overnight out of contact with the air did not develop a reaction rate materially different from the fresh sewage.

The difference between fresh sewage and that reaching the major treatment plants is not easily explained in simple chemical terms. Sulfide concentrations were not over one or two mg/l, and thus could not account for the rapid oxygen demand. Organic acid concentrations are higher in the old sewage, and it was at first supposed that this accounted for the high peak of oxygen demand and the sudden decline. The decline could be due to the exhaustion of a nutrient. It was found that addition of sodium acetate did indeed extend the rising curve, but the drop occurred long before the acetate was exhausted. It appears that the factors affecting the biological balance are highly complex, well deserving of further study, but too obscure at this time for fruitful speculation.

It is evident that the slime layer is a major source of an oxidizing culture suspended in the stream. There is no way to predict just how high the oxygen consumption rate might rise in a large sewerage system if it

were not constrained by oxygen starvation, because there has been no opportunity to observe a large system where oxygen impoverishment does not prevail. If adequate oxygen is supplied, and even if the reaction rate rises only as high as observed in the South Slope Trunk (Curve G of Figure 6 of Appendix I), a rather large degree of oxidation will occur.

EXTERNALLY GENERATED BIOLOGICAL FILM

Rather than relying on the sloughing of biological slime from the pipe wall to provide a suspended aerobic culture, activated sludge or trickling filter humus may be added. This may be feasible where an upstream treatment plant withdraws a portion of the flow from a trunk sewer, passes it through a secondary treatment plant to produce an effluent for local disposal, reuse, or groundwater recharge, and returns the waste sludge to the sewer. There are many upstream plants operating in this mode, but in no case has oxygen been added downstream for the purpose of stimulating in-sewer oxidation.

The capability of an upstream plant to generate sludge for in-line biological treatment is limited. If only a small fraction of the sewage flow is taken out and treated, the amount of activated sludge produced is not great enough to provide much enrichment of the culture in the sewer. If most of the flow is taken out, treatment of the residual flow in the sewer might be a minor adjunct to the main operation. If the flow taken out and treated upstream were equal to the flow that might be subjected to in-line treatment at some downstream point, the return sludge would be a substantial factor, even though the sludge concentration would be quite low in the comparison with the usual standards for mixed liquors in activated sludge tanks.

An alternative way to supply an oxidizing culture is to pump activated sludge from a downstream plant to a point far up in the system by way of an aerated pressure main. This pressure main would be an elongated "reaeration tank" of an activated sludge system. The downhill gravity

sewer, if it were kept aerobic, could then function as the mixed liquor tank.

A variation of this plan is to pump the entire flow in an aerated pressure main that would function as the mixed liquor tank. A final clarifier at the end of this pipeline would separate the sludge and return most of it to the sewer. This is a practical plan if there is an upstream point of use for the treated water. The Irvine Ranch Water District in Orange County, California, has been developing a project based upon this concept.

BIOLOGICAL CULTURE ON SOLID SURFACES

It has been shown in previous sections that the slime layer is an important source of an oxidizing culture in the stream. Under some conditions oxidation on the slime layer in place on the wall of the pipe is a major factor in the total process. The slime layer may be aerobic, anaerobic, or a mixture, depending upon the condition of the sewage. The outer zone of the slime layer, exposed to the sewage flow, contains aerobic organisms if there is even an intermittent exposure to dissolved oxygen.

Measurements have been made, principally in the course of preparing this report, of the rates at which the slime layer in sewers uses oxygen. Results are given in Appendix I. It was concluded that the slime layer frequently acts as an effective sink for all oxygen reaching it, in which case the rate of reaction is equal to the rate at which oxygen can be carried to the wall by the motion of the water.

The slime layer grown in a stream that is slow-moving but aerobic will attain considerable thickness, with long filaments trailing in the stream. The slime becomes progressively thinner as the velocity increases, but it was shown that it may still be a highly active layer. Even a slime layer so thin as to be unnoticed by casual visual inspection, in a sewer with a velocity as high as 2 m/sec (7 ft/sec), was able to use oxygen as fast as the stream supplied it.

Reaction of oxygen on the slime layer is predictable on the assumption that the slime is a sink for all oxygen reaching it. Where this condition prevails, the flux of oxygen to the pipe wall is proportional to its concentration, as shown by the following equation:

$$\phi_o = a[O_2]$$

where

ϕ_o = oxygen flux to pipe wall, g/m^2-hr

a = submerged-surface transfer coefficient, m/hr, defined by the equation

$[O_2]$ = dissolved oxygen concentration, mg/l or g/m^3

The rate of change of oxygen concentration due to the flux to the wall, designated R_o, is equal to the flux divided by the hydraulic radius:

$$R_o = \phi_o r^{-1} = a[O_2]r^{-1}$$

where

R_o = rate of change of oxygen concentration due to its utilization by the slime layer on the pipe wall

r = hydraulic radius (1/4 of pipe diameter in a filled pipe), meters

The proportional rate of loss of oxygen to the wall, designated the mural deoxygenation coefficient, is represented by the symbol K_o. It is related to the other quantities by the equation:

$$R_o = K_o [O_2]^{-1} = a\, r^{-1}$$

where

K_o = mural deoxygenation coefficient, hr^{-1}

The following empirical equation has been presented for estimating a (Appendix I):

$$a = 5.3(eu)^{1/2}$$

where

s = slope of the energy line of the stream

u = velocity, m/sec

Hence

$$K_a = 5.3\,(su)^{1/2}\,r^{-1}$$

The number of variables in this equation can be reduced by combining it with an empirical equation relating s, u and r. Best for this purpose, under turbulent flow conditions, is the Hazen-Williams equation, which can be written:

$$u = 0.850\,C\,s^{0.54}\,r^{0.63}$$

The Hazen-Williams coefficient for the pipe filled with sewage will be assumed to be 125.

$$u = 101.2\,s^{0.54}\,r^{0.63}$$

$$(su)^{1/2} = \frac{u^{0.93}}{(101.2\,r^{0.63})^{0.925}} = \frac{u^{0.93}}{71.6\,r^{0.58}} = \frac{0.0139\,u^{0.93}}{r^{0.58}}$$

$$(su)^{1/2} = 0.0140\,u^{1.43}\,r^{-0.58}$$

$$s = 0.076\,u^{1.43}\,r^{-0.58},\ \text{m/hr}$$

$$K_a = 0.074\,u^{1.43}\,r^{-1.58},\ \text{hr}^{-1}$$

$$R_a = 0.074\,u^{1.43}\,r^{-1.58}[O_2],\ \text{mg/l-hr}$$

Figures 14A and 14B provide graphical solutions to the equation for R_a.

FIGURE 14 A GRAPH FOR DETERMINING K_e (HIGH VALUES)

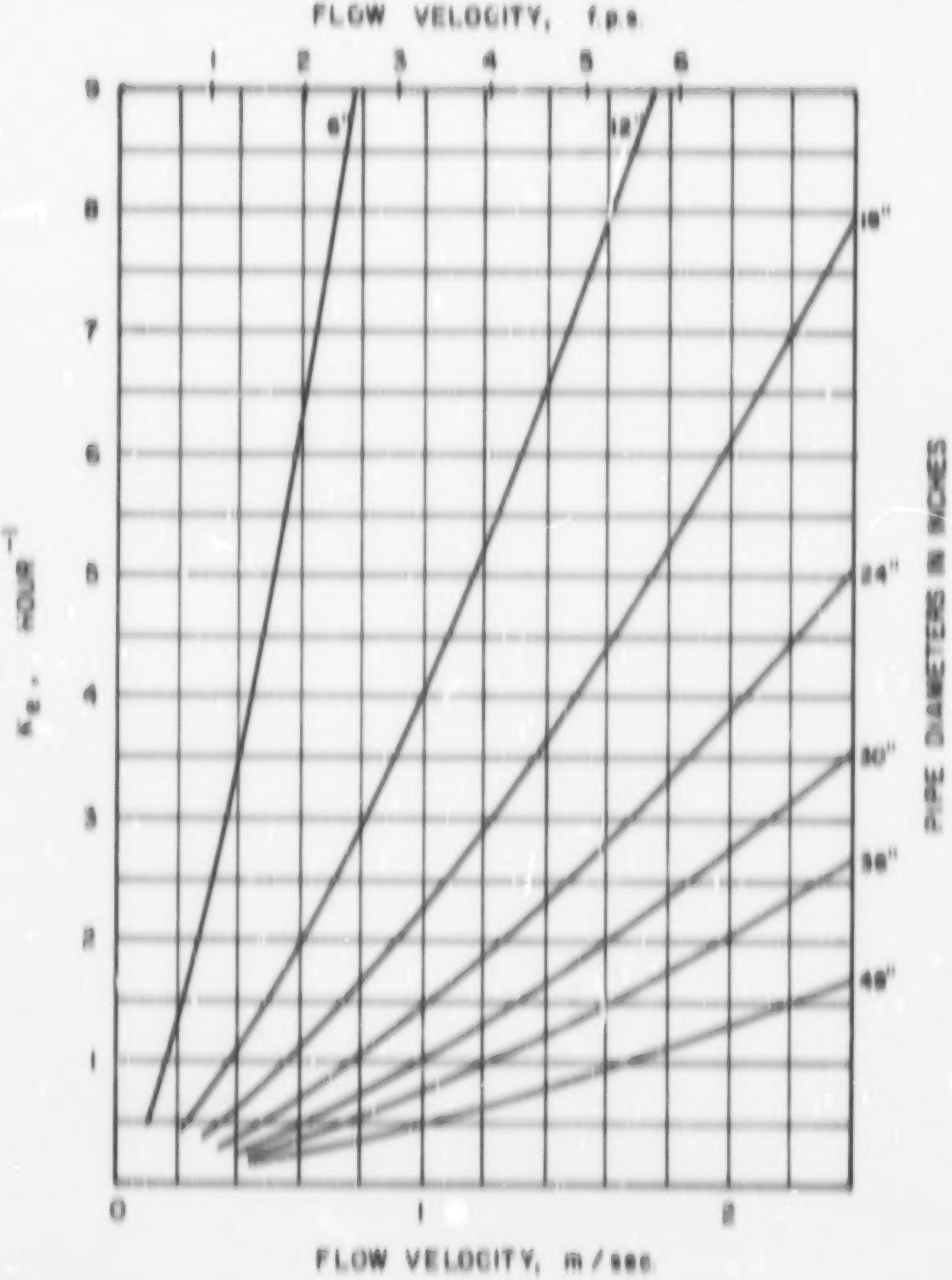
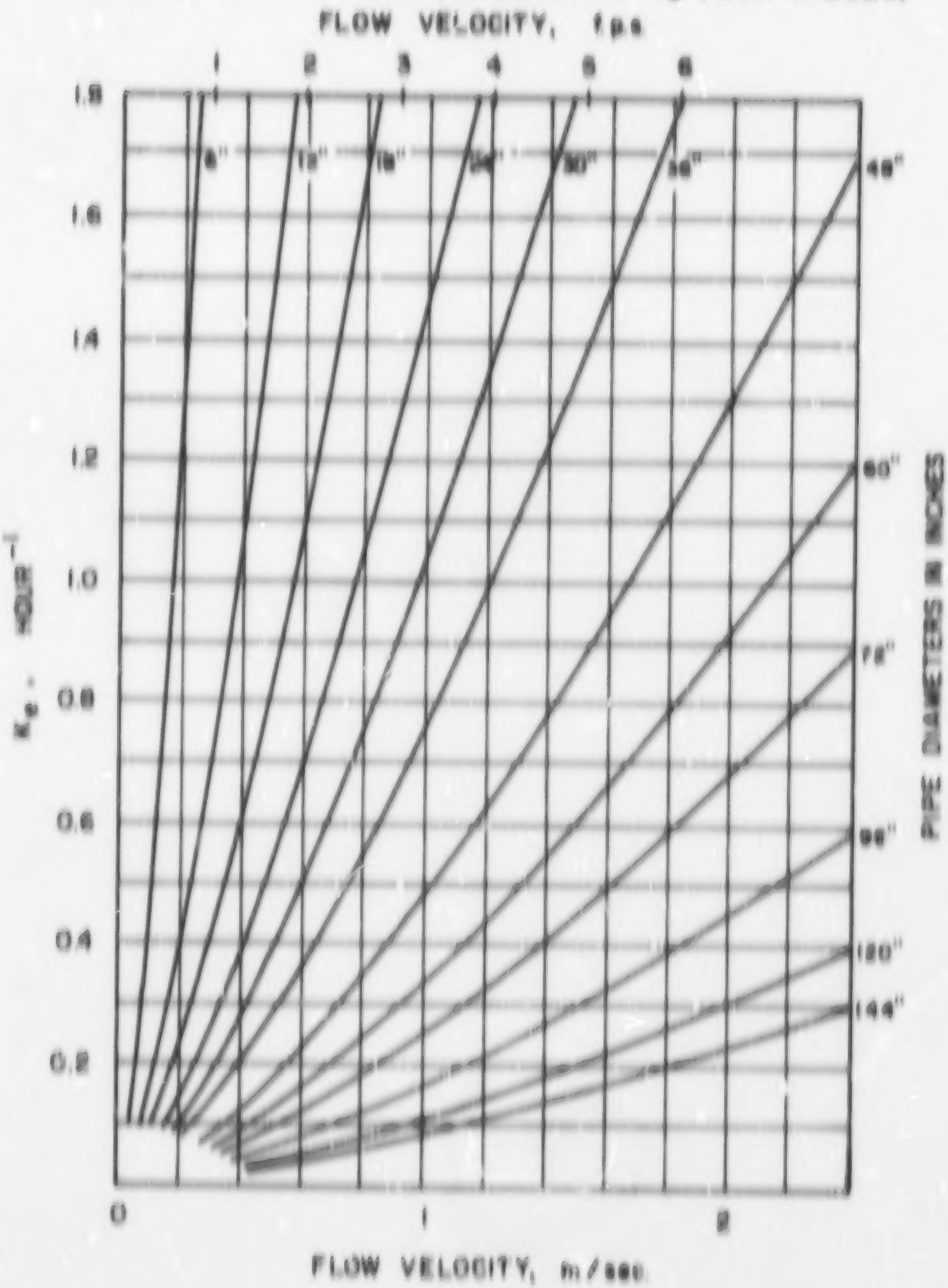


FIGURE 14B GRAPH FOR DETERMINING K_θ (LOW VALUES)



The proportional rate of loss of oxygen per meter of distance, designated K_e', is equal to K_e divided by the velocity.

$$K_e' = \frac{0.0/A}{3600} u^{0.43} r^{-1.58} = 0.000026\, u^{0.43} r^{-1.58}$$

where

3600 = conversion factor from seconds to hours

This equation shows that in a pipe of given length the stream will lose more oxygen to the pipe wall when it is moving at a high velocity than at a low velocity, despite the shorter transit time. However, because of the shorter time the amount used by reaction in the stream will be less.

In a reach in which there is no oxygen input, as in a pressure main, the rate of change of oxygen concentration is given by the expression

$$-\frac{d[O_2]}{dt} = K_r + K_e$$

where

$-\frac{d[O_2]}{dt}$ = decline of oxygen concentration, mg/l-hr

K_r = rate at which oxygen is depleted by reaction in the stream

K_e = rate at which oxygen is depleted by reaction on the pipe wall

Assuming K_r to be constant, the equation can be integrated, with the following result:

$$t_2 - t_1 = \frac{2.303}{K_e} \log \frac{K_r + K_e[O_2]_1}{K_r + K_e[O_2]_2}$$

where

t_1 and t_2 = times in hours

$[O_2]_1$ and $[O_2]_2$ = corresponding oxygen concentrations

General curves can be drawn from this equation for any combination of K_r and K_e values. $[O_2]_1$ may be given any appropriately high value. For

any lower value of $[O_2]$ the curve is the same from the point representing that initial concentration.

EXAMPLE A

Sewage will be pumped at a velocity of 0.61 m/sec (2 fps) through a force main 1.83 m (3 ft) in diameter. The oxygen reaction rate of the sewage is assumed to be 11 mg/l. Assume that the initial oxygen concentration is 40 mg/l, calculate the times when it will be 30 mg/l, 20 mg/l, 15 mg/l, 10 mg/l, 5 mg/l, and zero.

EXAMPLE B

Sewage of the same composition as in Example A will be pumped through a pressure main 0.305 m (1 ft) in diameter at a velocity of 1.28 m/sec (4.2 fps). Calculate the times required to reach the same oxygen concentrations as in Example A.

From Figures 14 A and B, R_0 is 0.165 in Example A, and 5.7 in Example B. Figure 15 is a graphical representation of the solutions. In Example A, the initial R_0 value is 6.6 mg/l-hr and $(-d[O_2])/(dt)$ is 6.6 + 11.0 = 17.6 mg/l-hr. Thirty-seven percent of the initial oxygen consumption rate is due to reaction on the slime layer. In Example B the initial rate of oxygen consumption is calculated to be 239 mg/l-hr, of which 95 percent is due to usage by the slime layer.

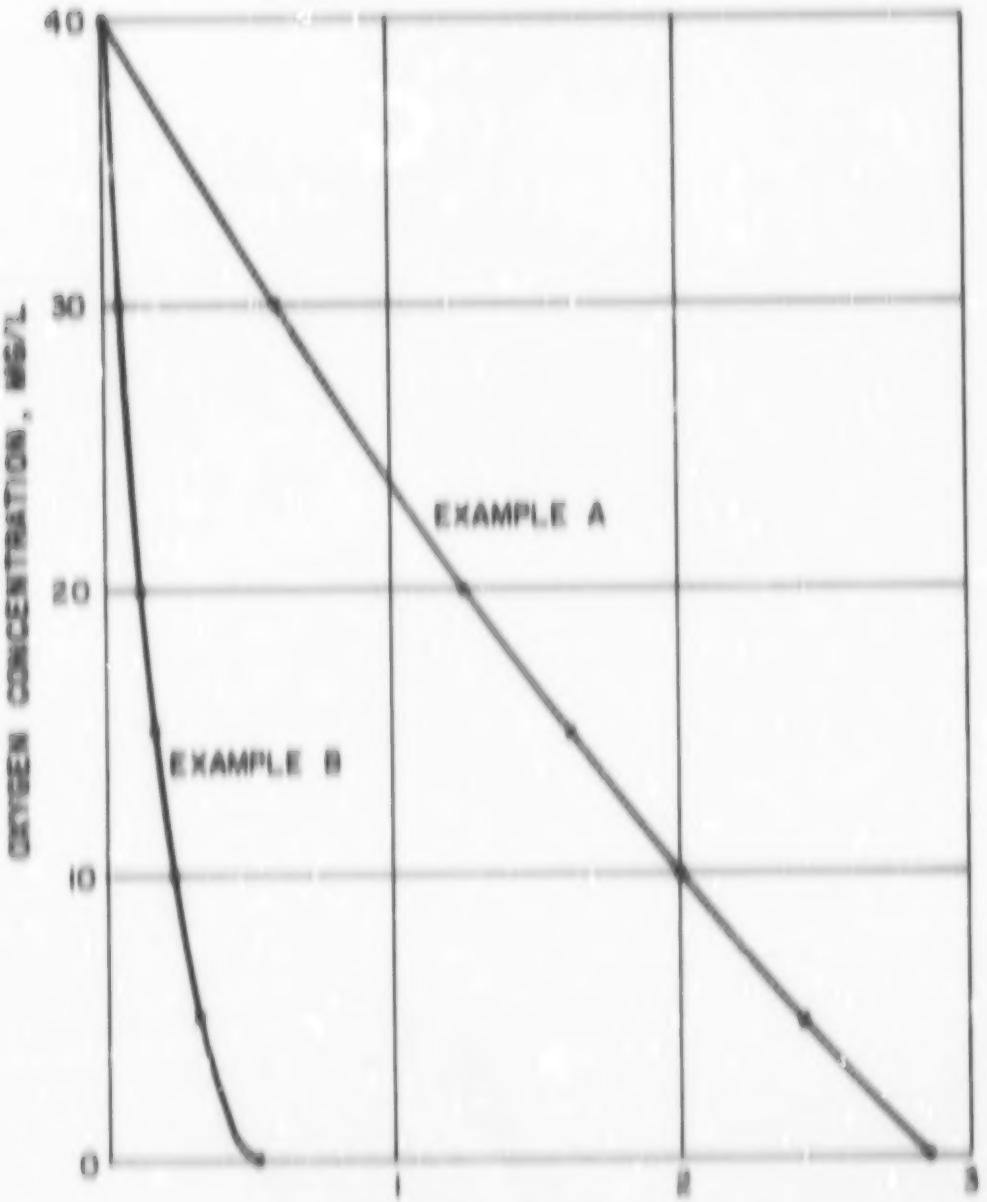
The prediction of such a high rate of oxygen usage by the slime layer must be treated with caution. The flux of oxygen to the pipe wall, ϕ_0, is given by the equation

$$\phi_0 = sK_0[O_2]$$

In Example B the initial flux is calculated to be

$$\phi_0 = 0.0762 \times 5.7 \times 40 = 17.4 \text{ mg/m}^2\text{-hr}$$

FIGURE 15 SOLUTIONS FOR EXAMPLES A & B



CALCULATED OXYGEN DEPLETION CURVES

The highest flux that has been measured in a sewer is 2.1 mg/m^2-hr (Appendix I). Since that was in a sewer in which the velocity was 2.16 m/sec, it appears that velocity is not a limiting factor under the conditions assumed in the examples. It is possible that the flux might be limited by the metabolic capability of the thin biological layer that would be supplied by oxygen diffusing into it. It seems quite likely that flux rates as high as 20 g/m^2-hr could be sustained, but it is a question that can be answered with assurance only by experimentation. A further limitation of the flux may be the supply of oxygen-demanding nutrients. If these nutrients do not reach the slime layer at a rate matching the oxygen supply, then it will be these nutrients rather than the oxygen supply that will control the rate of oxygen usage. All things considered, it seems likely that oxygen consumption in a small pipe with fairly high velocity and high oxygen concentration may be sufficient to accomplish the equivalent of secondary biological treatment in a period of an hour or two.

In large trunks, longer times will be necessary. In some regional systems being planned, the times may be long enough to carry the oxidation well along toward completion, but in most cases it will be a matter of partial oxidation, reducing the load on the sewage treatment plant. The equations that have been presented will suffice for the development of designs that will optimize the contribution that major sewers will make toward the final treatment objectives.

INCREASING THE SLIME-SUPPORTING SURFACE

Since all surfaces submerged in the stream will accumulate slime, the question naturally arises as to the feasibility of increasing the surface area, thus augmenting the "trickling filter effect." There are several ways that this could be done but all of them present practical difficulties.

The evident greater efficiency of oxidation gained in smaller pipes suggests minimizing pipe diameter. If there were two practical pipe sizes for a force main, roughly equal in respect to all other determining factors, or if the alternatives of one larger pipe vs. two smaller ones were fairly well balanced, and if it were considered that in-sewer BOD reduction would be a desirable objective, then the smaller pipes would be chosen. Such situations will be rare.

Slime-supporting area could be increased by fitting one large pipe with a number, perhaps 3, 6 or 7, of smaller pipes of light-weight plastic. Seven small pipes would increase the slime area more than fivefold, but at a high installation cost and with a tripling of the friction head.

A gravity (partly-filled) sewer might have the lower half lined with a plastic sheet having vertical vanes projecting into the stream, aligned parallel to the axis of the pipe. This arrangement, too, would be high in cost in comparison with any benefits gained.

CHAPTER 6
CHEMICAL TREATMENT

POTENTIAL APPLICATIONS

Applications of chemicals to sewage streams represent the oldest form of in-line sewage treatment. The principal objective of these treatments has been for the control of sulfide, although various other benefits have been claimed, such as removal of grease, keeping the sewer clean, BOD reduction, etc. The chemical most widely used in sewers has been chlorine, applied since the 1920's. It destroys any sulfide and mercaptans that may be present, depresses oxygen consumption so that an oxygen reserve accumulates in the stream, and, in large doses, may inactivate sulfide-producing slimes. Iron salts, zinc salts, calcium or sodium nitrate, and shock doses of calcium or sodium hydroxide have also been successfully applied. Hydrogen peroxide has been introduced recently for sulfide control, providing a source of oxygen, expensive but sometimes favored because of the simplicity of application. Not so successful have been chlorinated hydrocarbons, chlorinated phenols, ozone, and various "patent medicine" products of unrevealed composition. Sulfide control, however, is not the subject of this report. Chlorine presumably does reduce the chemical oxygen demand by the oxygen equivalent of the chlorine added, which, being only 0.225 of the chlorine dosage, generally does not amount to much. Peroxide accomplishes biological oxidation to the extent of the oxygen released, which is 47 percent of the weight of H_2O_2. Oxygen itself, applied either as air or as refined oxygen, is an essential element of biological treatment, but that subject has been covered and is not classed as chemical treatment.

The principal types of chemical treatment that need to be looked into for possible in-sewer use are those that produce precipitates capable of entraining or adsorbing or otherwise precipitating impurities from the water. In both water and wastewater treatment plants use is made of lime, aluminum sulfate, and long-chain organic polymers or polymerized silicic

acid (activated silica). The polymers are used mostly along with aluminum sulfate as coagulant aids, but sometimes acting alone as coagulants. Iron salts are sometimes chosen in place of aluminum sulfate for treatment of wastewaters. The iron and aluminum salts behave similarly, and do not need to be treated separately for the purposes of this discussion.

The successful use of iron or aluminum salts depends upon flocculation of the initially formed precipitate, which requires gentle mixing for a period of time. It is this function that may be performed in the sewer, thus saving the costs for construction and operation of flocculation equipment.

The development of a good floc depends upon the proper intensity of stirring. This intensity can be measured as a velocity gradient, which is equivalent to the square root of the rate of energy loss per unit mass of water divided by the dynamic viscosity of the fluid. The mean velocity gradient, in units of sec^{-1}, is given by Camp (20) as

$$G = \frac{du}{ds} = \sqrt{E \div \nu}$$

where

$\frac{du}{ds}$ = rate of change of velocity (u) with distance (s)

E = energy dissipation rate per unit of volume

ν = dynamic viscosity

For flow in a pipeline, E is proportional to slope (s) times velocity (u). If velocity is expressed in feet per second, G has these values:

At 10°C $G = 1980 \sqrt{su}$

At 20°C $G = 1730 \sqrt{su}$

At 30°C $G = 1550 \sqrt{su}$

From observations in water treatment plants, Clark (21) concluded that suitable values of G for good flocculation are in the range of 20 per second to 70 per second. Similar conclusions have been reached by others.

Figure 18 can be used for obtaining values of G, as well as velocities, from slopes and quantities of flow. The graph is applicable only in partly filled pipes in which the flow is between 1 and 99 percent of full-pipe capacity. It has been shown that within this range the velocity is practically independent of pipe diameter (22). The same would be true for G.

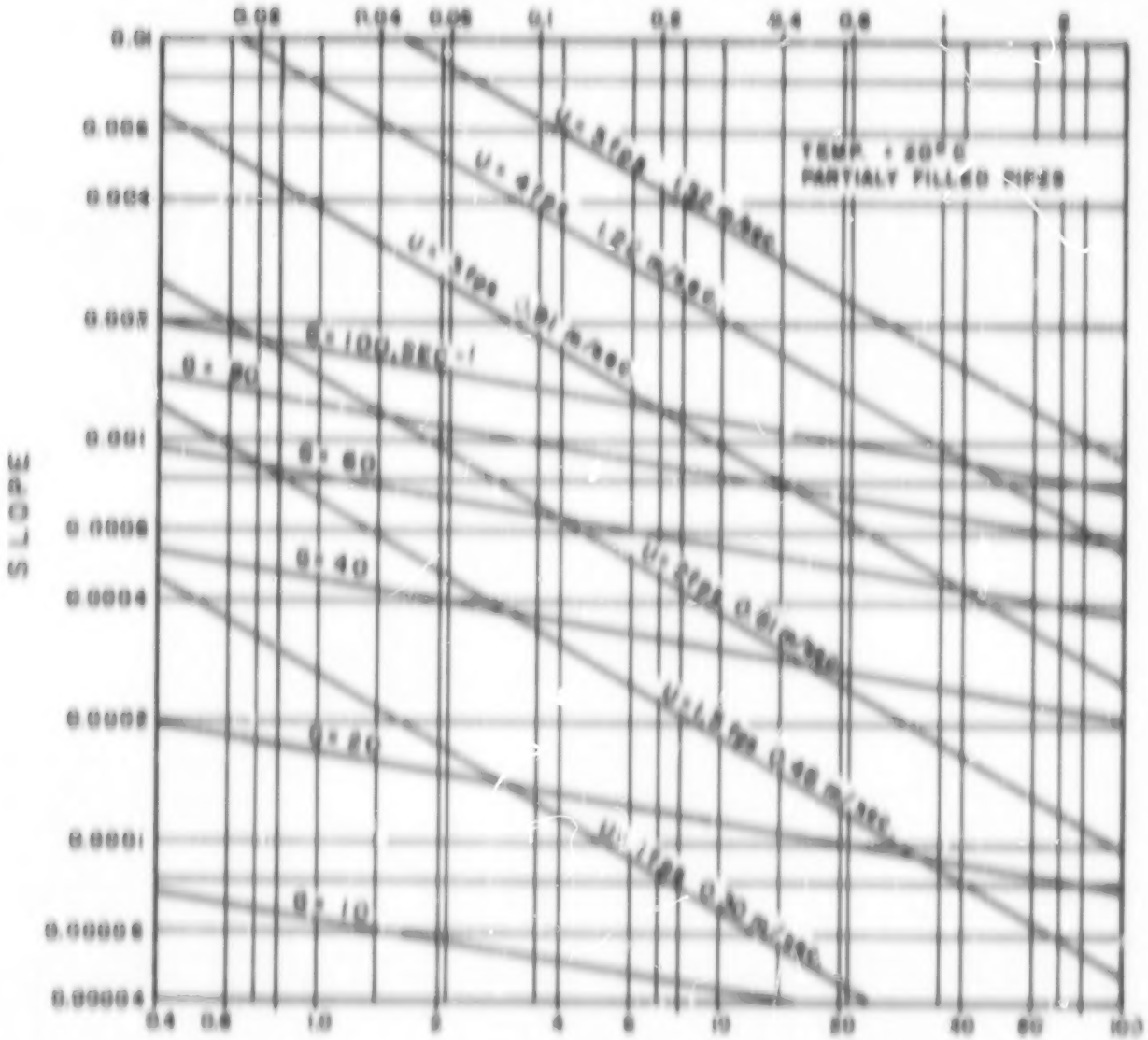
It is evident from Figure 18 that velocity gradients found in sewers are quite often in a range considered suitable for flocculation. The requirement for self-cleansing velocities eliminates G values that are too low. Values that are too high, however, may be found in large swift streams.

Ideally, the dissipation of mixing energy should be distributed uniformly in the flocculation chamber. This ideal can never actually be attained, and the departures from the ideal may be large. Flocculators with similar average velocity gradients may perform differently, depending upon how well the shear or velocity gradient is distributed. The turbulence due to flow in a pipe will presumably come much nearer to an ideal distribution of the mixing energy than will the turbulence produced by paddles. Good flocculation may be obtainable at higher G values than in conventional flocculators.

It is necessary that the flocculation process be continued for a suitable length of time. It is considered by some engineers that the dimensionless product Gt is a useful parameter for design. Camp (20) found that good operation has been obtained with Gt values between 20,000 and 200,000. However, the time should not be less than ten minutes. Suitable flow times for the attainment of good flocculation will probably be between 10 and 30 minutes in most cases.

FIGURE 16
VALUES OF VELOCITIES, u, AND VELOCITY GRADIENTS, G,

FLOW, CUBIC METERS PER SECOND



FLOW, CUBIC FEET PER SECOND

Another requirement for the most efficient treatment with coagulating chemicals is that the initial mixing of the chemical into the water be very rapid. Addition of the chemical at a location where the stream enters a drop of two feet or more would result in good initial mixing. Otherwise a mixer with an energy input of one to two horsepower per cfs of flow should be provided. The volume of the mixing chamber need be only large enough to absorb this energy input.

The greatest obstacle to in-sewer flocculation is the requirement that the sewage then be transferred to the settling basin without floc disrupting turbulence. Where the sewage must be lifted into the treatment plant by pumps, as is true in most plants, in-sewer flocculation would be fruitless. Even where the sewage is not pumped, the floc could be preserved only by avoiding any hydraulic jump, high velocities in pipes, or excessive turbulence created by valves or other fittings. It would, therefore, be required that the level of water in the primary clarifier be practically the same as in the sewer. Since the elevation in the sewer varies, the level in the primary tank would need to be varied to keep pace. A control on the discharge of the tank would serve this purpose. It would not be harmful if the water were backed up somewhat in the sewer on the lowest flows, so that the range of water surface elevation in the tank could be less than the normal range in the sewer under uniform flow conditions. For small sewers, up to, say, 0.6 m, no special problem would be created. With large trunks, however, it would become difficult to operate a constant-level clarifier while at the same time preserving a floc produced in the sewer. A 1.5-m diameter trunk, as an example, would require a range of at least 0.3 m and perhaps 0.6 m feet in the clarifier. Conventional clarifiers are not built to accommodate such fluctuations, but they could be. It is not necessary that the water be taken from the tank by falling over a weir. Tanks with submerged withdrawals are used in a number of places, the level being modulated by a float controlled valve or a float controlled pumping rate out of the clarifier. In one case a secondary small weir is used. Such a weir could be made adjustable. There is much to be said in favor of weirless tanks, especially where odor release is a problem.

The skimming of a variable-level tank is another problem that could be solved with a little ingenuity, as for example by a floating skimming boom and a floating trough.

It is doubtful that a comminutor or bar screen could be made large enough to avoid turbulence that would disrupt a flocculent precipitate; it is possible, however, to dispense with such devices ahead of primary settling, using relatively large pumps of suitable design to remove the sludge to a shredder, degritter, thickener, or such other devices as may appear necessary.

The incentive to develop designs to bring the sewage into primary tanks without passing through a zone of rapid energy input or dissipation should not be confined to instances of planned chemical flocculation. In many cases the suspended solids arriving at a treatment plant are more or less flocculent, as a result of either chemical or biological processes occurring in the sewer. Insofar as that flocculent character can be preserved, better clarification will result.

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APPENDIX I

SELF PURIFICATION IN SEWERS

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The oxygen balance in a stream of sewage is important in respect to the possible generation of sulfide, and in respect to the degree of biological oxidation that may occur in a sewer.

In a previous paper [1], the equation for oxygen balance in a stream of sewage was written

$$\frac{d\mathbf{D}}{dt} = R_r + R_s - R_f \qquad mg/l\text{-}hr \qquad (1)$$

in which $\mathbf{D}$ = oxygen deficit, g/m³ (mg/l)

t = time, hours

R_r = rate of reaction of oxygen in the fluid, g/m³-hr

R_s = rate of loss of oxygen to slimes on the pipe wall, g/m²-hr

R_f = rate of supply of oxygen to the stream by surface aeration, g/m³-hr

R_f is proportional to $\mathbf{D}$, and may be replaced by

$$R_f = \frac{f\mathbf{D}}{d_m} \qquad (2)$$

in which f is designated the "exchange coefficient," expressed in m/hr, and d_m is the mean hydraulic depth in meters.

In the article cited, a predictive equation for f, applicable to sewage streams, was developed:

$$f = 0.96(1 + 0.17\,\mathbf{F}^2)\,\gamma\,(su)^{1/2} \qquad (3)$$

$\mathbf{F}$ is the Froude number, $\sqrt{\frac{u}{g\,d_m}}$; γ is a temperature function, equal to 1.00 at 20°C; s is the slope ratio; and u is the velocity in m/sec.

If f is expressed as feet per hour and u as fps, the coefficient is changed from 0.96 to 3.0.

With the aid of the predictive equation for reaeration, attention will now be given to R_r and R_s.

C/31/11/1

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Reaction Rate of Oxygen in Sewage - Laboratory Tests

In field work, a sample is taken from a sewer in a "D.O. pot" of the design shown by Parkhurst and Pomeroy [1]. After brief aeration, the diaphragm, carrying the mixer, thermometer and oxygen probe, is put in place, and oxygen concentrations are read over a suitable interval of time, yielding data for calculating R_t. In some of the laboratory determinations, use was made of a liter beaker and a light rubber diaphragm, with a water-powered magnetic stirrer underneath so that the whole assembly could be placed in a water bath. Comparative tests showed that the speed of mixing does not make any measurable difference as long as it is sufficient to keep the organic solids suspended, and provided also that the water velocity past the probes is sufficient for reliable oxygen results when the micrometer is read. Fig. 1 shows typical oxygen depletion curves.

Within the limits of accuracy of the measurements, the reaction rate appears to be independent of oxygen concentration until the oxygen concentration is quite low, generally a few tenths of a milligram per liter. Kessler and Nichols [2], quoting unpublished work by Carl H. Nordell, first reported that oxygen usage in activated sludge is essentially independent of oxygen concentration. They showed straight lines extending into a region of negative oxygen concentrations, construed as the development of substances of essentially instantaneous oxygen demand.

To explore this phenomenon further, the sewage in a D.O. pot was allowed to stand anoxic for a half hour. A measured amount of a solution of 35 mg/l of oxygen in water was added by way of a syringe in a suitable amount to produce a concentration of

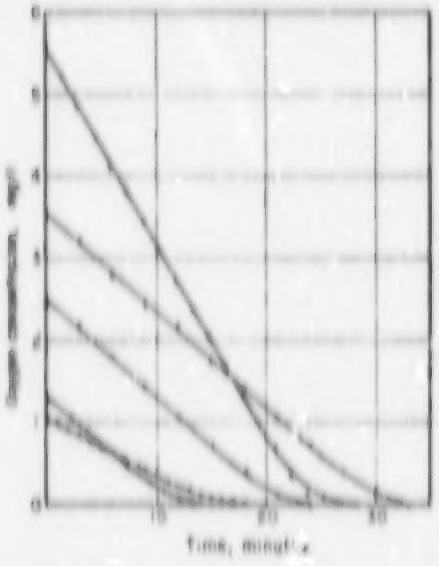


Fig. 1
Typical oxygen consumption curves

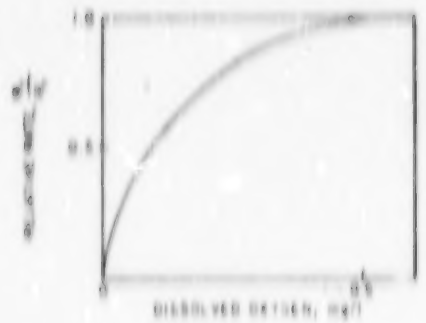


Fig. 2
Typical relationship between low oxygen concentration and oxygen reaction rate.

1.38 mg/l in the sewage. The earliest dependable reading of the probe was 1½ minutes after the start of addition of oxygen solution; it showed 1.23 mg/l. The subsequent reaction rate was 0.06 mg/l-minute. Within the limits of accuracy of the measurements, it appears that the reaction had merely come to a dead stop, going on as soon as oxygen was again supplied.

Anaerobiosis can produce substances exerting a chemical oxygen demand, sulfide and ferrous iron being obvious examples, but when these compounds are present in small amounts, their rates of utilization of oxygen are not large. The results of Kessler and Nichols may have been due not to the development of an avidity for oxygen, but to interferences in the Winkler test used to determine oxygen, causing all oxygen measurements to be low.

The rate of reaction of oxygen in the high oxygen demand region is designated R'_t. When R_t/R'_t is plotted against dissolved oxygen concentration, one obtains curves of the general shape shown in Fig. 2. It would be difficult to account for the course of the curve on the basis of a chemical reaction mechanism. The lag of the probe as oxygen reaches zero could contribute to the curvature, but this effect is small and would not account for the shape of the curve. More likely the process is diffusion-controlled when oxygen concentrations are small. The biologically active material is mainly flocculent. Oxygen must reach the interior of floc particles by molecular diffusion. At low oxygen concentrations, the cores of flocs become starved for oxygen, and as these anoxic nuclei grow in size, the amount of oxygen used by the sewage per unit of time decreases. This view is supported by the observation that oxygen depletion rates determined on samples with rather coarse suspended matter show curvature at higher oxygen concentrations than do samples lacking coarse solids.

Oxygen reaction rates were determined on samples from the Los Angeles County Sanitation Districts system. The standard 5-day BOD, and even the 1-day demand, bear little relationship to the 1-hour demand, as shown by Fig. 4. The rates there shown have been corrected to 20°C on the assumption that $\frac{R_t}{R_{t\,20}} = 1.07^{(T-20)}$.

Results from various test sites are given in Fig. 4. The designations of the sampling sites, shown at the bottom, consist of numbers followed by letters. The numbers signify the sewer pipe diameters in inches. There is a tendency for R_t to be lower in small sewers than in large ones. It was found that R_t usually increases as the sewage flows along.

The sewage from test site 24C has an abnormally low value. It was found that the pH of that sewage was quite high much of the time, up to 10.0, thus suppressing biological activity. The high pH is due to the frequent discharging of highly alkaline industrial wastes, a practice encouraged by the Sanitation Districts because of the beneficial effect in suppressing sulfide generation.

Two series of tests, one in September and one in January, were made along Los Angeles County Sanitation Districts South Slope Trunk, on the seaward slope of the Palos Verdes Hills (Fig. 5). The sewage was timed by use of dye, and the testing paced the flow. Flow is essentially by gravity, but there are two siphons, and there is a 5,400-ft length of above-ground steel pipe of indefinite profile, laid over a creeping landslip. Two lift stations pumping intermittently into the line were shut off during the January tests. They were not shut off during the September tests, but it appears that they had little, if any, effect; they may not have contributed to the sewage tested.

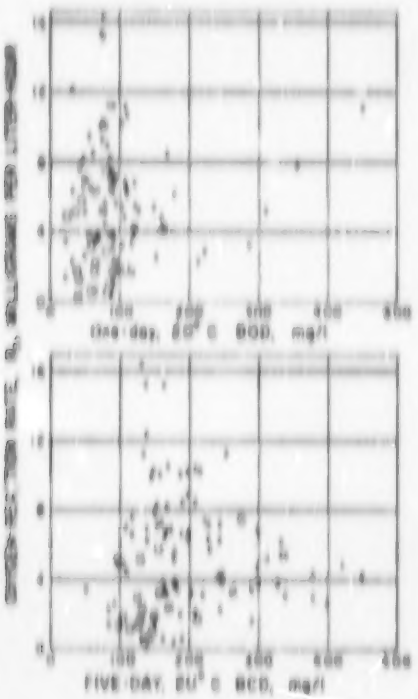
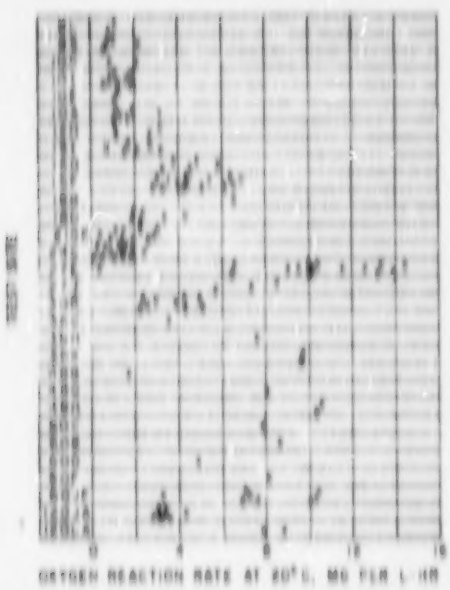


Fig. 4
Comparison of oxygen reaction rate with 1-day and 5-day BOD. All results corrected to 20°C. BOD on settled samples indicated as circles, BOD on whole samples indicated as dots. All R_t tests were on whole sewage.



OXYGEN REACTION RATES

Fig. 4
Oxygen Reaction Rates

Tributary sewers brought in sewage that always had lower R_t rates than the mainline flow, producing a saw-tooth profile of R_t values. The circled points in Fig. 5 are the only ones actually measured on the main line. The other points are calculated from the flows and the measured or assumed values for the characteristics of the influxes. In the case of the dissolved oxygen graphs, the calculated mixtures may be considerably in error, because turbulence at the junctions doubtless carries additional oxygen into solution. A part of the incremental flows is due to service connections along the route. These contributions, too, produce aeration as they enter the mainstream. Thus, dissolved oxygen is present throughout most of the reach, even though R_t generally exceeds the calculated reaeration rate, tabulated at the top of Fig. 5. The oxygen supply rates, incidentally, are not greatly affected by temperature, because the increase of the reaeration coefficient with temperature has an effect opposite to the decrease of oxygen solubility.

After leaving the South Slope Trunk, the sewage enters the Whites Point pump station, along with an additional tributary flow. It spends 45 to 60 minutes in the force main, being kept aerobic by injection of air. Fifteen pairs of offset samples at the pump station and end of the force main showed that R_t sometimes increases in the force main and sometimes decreases. The average rate at the pump station was 16.4 g/m^3-hr, and 17.6 at the discharge end.

The increase of R_t as the sewage proceeds downstream, shown as well in the Palos Verdes South Slope Trunk, seems to be a normal characteristic of sewage near its source, continuing for a considerable distance if oxygen is supplied at a sufficient rate. The South

Slope Trunk is not unusually well aerated, but it drains a hilly terrain, so that junctions tend to keep it from becoming anoxic along most of its length. Sewers in flat areas, laid at minimum slopes, are likely to provide adequate aeration only where the flow depth is shallow. The deeper flows in larger trunks become nearly anoxic.

Various sewages were aerated in the laboratory, and samples were withdrawn from time to time for R_t determinations. Fig. 6 shows typical curves for the changing reaction rates. Curves A and B were made with samples of strictly residential sewages taken near the points of origin. Curve C was obtained with sewage from the South Slope Trunk at Whites Point pump station. Even though it had spent an average of only about 16 hours in the trunk, it had passed the R_t peak. Curves D, E, and F are for sewages from three metropolitan trunks entering the Los Angeles City plant at Hyperion and the Los Angeles County Sanitation Districts plant at Harbor City. These sewages probably had average ages of about 10 hours, but for most of that time they had carried very little dissolved oxygen. The small secondary rise shown in curve E is not unique. It was clearly shown in several other sewages, especially as the techniques of measurement were refined.

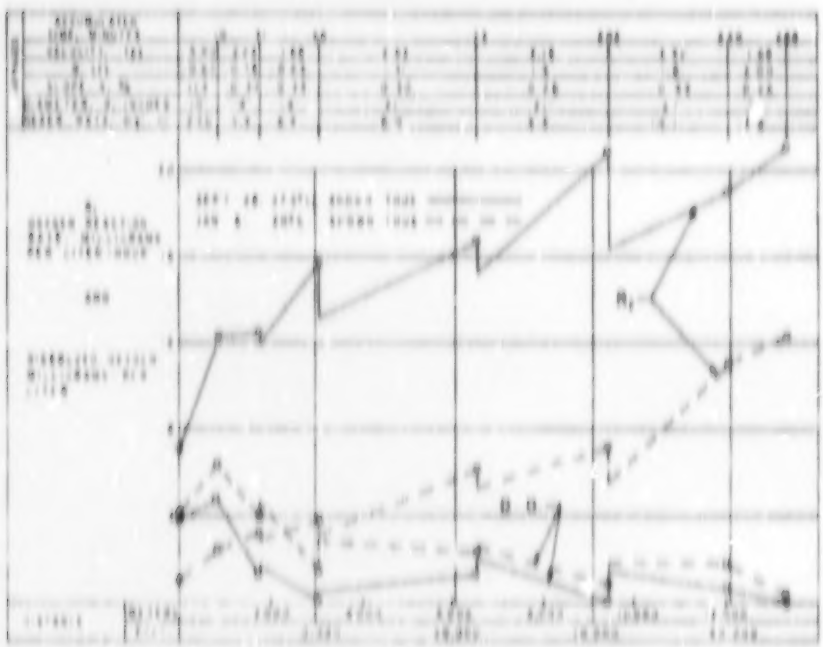


Fig. 6
Oxygen balance, Palos Verdes South Slope Trunk

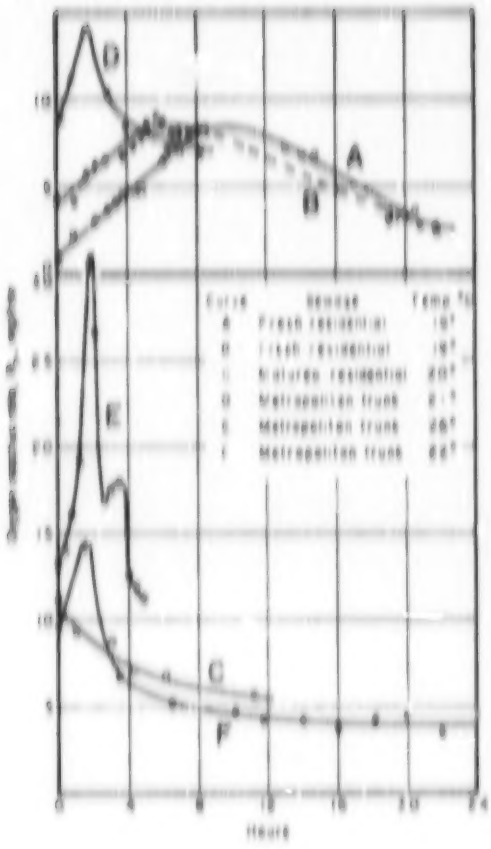


Fig. 6
Changing R_t values in aerated sewages

The pronounced peak in the oxygen demand rate curve is intriguing. On a guess that organic acids might be a factor, duplicate samples of a residential sewage were run in parallel, but to one sodium acetate was added to a concentration of 44 mg/l as acetic acid. The test was run at 27.5°C. R_t increased slightly faster in the one with added acetate. The striking difference appeared when the untreated sample reached a peak of 27 mg/l-hr at 2.5 hours and then declined abruptly, but in the presence of acetate R_t climbed for 1.0 hour more, to a peak of 47 mg/l-hr. The difference between the amounts of oxygen used by the two samples up to the point when the second one declined was 22 mg/l, or 0.95 mol of O_2 per mol of acetic acid. Considering only the stoichiometry, one might suppose that the acetic acid was oxidized to glyoxylic acid. A similar experiment with another sewage showed a mol ratio of 0.9.

A sample that had had acetate added reached an R_t peak of 38 mg/l-hr at 27°C. After aerating it for an hour more, it was allowed to stand overnight without air and then was aerated six hours more, at which time R_t was 6.2. Acetate was again added in the amount of 70 mg/l. Immediately thereafter R_t was 42 mg/l-hr at 26.3°.

The strong evidence that acetate oxidation accounts for the peak was clouded by a test run on sewage from the source that gave curve E in Fig. 6. A sample was analysed for volatile organic acids by distillation and titration, yielding a result of 0.76 me/l. The sewage was aerated until the peak was passed, then analysed again. The organic acid content was the 0.35 me/l. Gas chromatography of the acids before and after oxidation showed the proportions by equivalents to be 63% and 61% acetic, and 19% and 17% propionic. Small amounts of the butyric, valeric and caproic acids made up the remainders.

The findings from these exploratory experiments may be sufficiently challenging so that others will pursue the subject further. The results have a bearing on oxygen utilization in sewers, and on the possibilities of in-sewer treatment, and may throw light on the apparent differences of treatability of sewages of different ages and in different climates, and on the effects of digester liquors returned to the activated sludge process.

The biological oxidizing culture develops more rapidly in the sewer than it does in an aerated vessel in the laboratory. In some sewers the culture tends to be flocculent in character, settling rapidly and leaving the sewage with a low R_t value, but in other cases it settled poorly. The samples aerated in the laboratory settle very poorly. It appears that the biological floc grown in the sewer is, in part, slime that has sloughed from the pipe wall.

R_t can be determined from oxygen balance data in sewers, provided R_s is reduced to zero. This was done by adding sodium hydroxide to the sewage stream to raise the pH to 12 or above for about a half hour followed by sodium hypochlorite added at high concentration to assure that the biological slime layer was not only killed but also well oxidized. Hypochlorite treatment alone did not suffice unless continued at a high dosage for a long time. After the cessation of chlorination, oxygen concentration was measured at both ends of a test reach, along with measurements of velocity, discharge and temperature. The test procedures were as described previously. The oxygen exchange coefficient, f, was calculated by Eq. 3. Since the changes of oxygen concentration in the reaches were usually small in comparison with the average deficit, the differential equation was usually replaced by the approximation

$$-\frac{\Delta(D_2)}{\Delta t} = R_t - \frac{f(D_2)}{d_m} \qquad (4)$$

in which (D_2) is the oxygen concentration in mg/l.

Results are shown in Table 1. It is believed that the largest source of error causing discrepancies between the values of R_t obtained by the two methods is in the determination of $\Delta(D_2)/\Delta t$, especially when the change in a time as short as 7 minutes must be converted to a per-hour rate. There appears to be a tendency for the sewer results to be lower than the pot tests, but the difference is not statistically significant.

Oxygen Consumption by Slimes

Oxygen balance tests in sewers in their normal condition, together with determinations of R_t on samples withdrawn from the sewer, allow estimates of oxygen usage by the slime layer.

Table 1: Oxygen Reaction Rates Determined in Sewers

Table 2 shows results of 48 tests in 8 sewers. Results for each sewer were averaged, except that when a sewer was tested under two quite different flow conditions, or at two times separated by an interval during which the hydraulic condition changed because of cleaning or some other circumstance, the results were divided into two groups. Thus Table 2 shows averages of ten sets of runs.

It may be noted that $\Delta(D_L)$ was negative in all cases. The reaches tested were necessarily the maximum available distances presenting uniform conditions, with no junctions. In all cases there were junctions not far upstream, resulting in points of turbulence raising dissolved oxygen to levels that could not be sustained by the reaeration rates in the test reaches.

It seems reasonable to suppose that the net rate at which the slime layer can use oxygen will be influenced by oxygen concentration. In view of the rich food supply, the slimes may develop a capacity to use oxygen as fast as it arrives. If this is true, then the amount of oxygen used should be proportional to the oxygen concentration in the water.

The flux of oxygen to the slime layer, ϕ_0, is equal to $R_0 r$, in which r is the hydraulic radius. If the flux is proportional to the oxygen concentration (D_L), one may define a transfer coefficient, e:

$$e = \frac{\phi_0}{(D_L)} = \frac{R_0 r}{(D_L)} \qquad (5)$$

e is analogous to the exchange coefficient from air to water, and will be expressed in oxygen balance calculations as m/hr.

If the rate at which the slime layer uses oxygen is dependent upon the rate of arrival of oxygen, then it should be proportional not only to the concentration of oxygen in solution, but also to the rate of renewal of water adjacent to the slime layer by eddy diffusion. Thus, it may be a function of the rate of energy dissipation. If it be assumed that it is a simple power function, one may write

$$e = C_E (wr)^n \qquad (6)$$

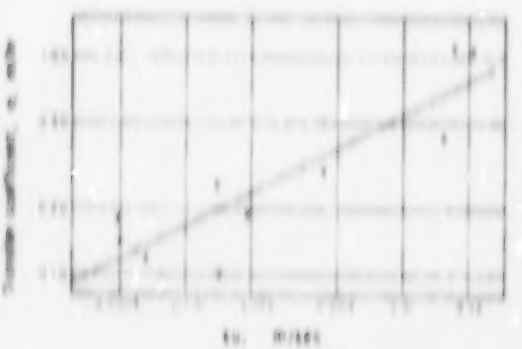


Fig. 7
Effect of rate of energy dissipation on coefficient for transfer of oxygen to the slime layer.

Fig. 7 shows a plot of e vs. su. The regression line indicates q = 0.51. The standard error of the exponent is about 0.11. The data are not free from internal correlations; the uncertainty of the exponent, therefore, is greater than indicated by this standard error. Taking q to be ½ and determining C_3, the equation becomes

$$e = 5.3\,(su)^{1/2} \quad \text{(e in m/hr, u in m/sec)} \qquad (7)$$

In filled pipes, e varies approximately as u^1. Hence $(su)^{1/2}$ is proportional to $u^{3/4}$. Heat transfer to or from the wall of a pipe is generally held to vary as u^{3}.

Eq. 7 applies only where the slime is an effective oxygen sink. In a well aerated stream of low oxygen demand, the substrate may be entirely aerobic, in which case the rate of usage by the slime layer would not be proportional to the oxygen concentration.

One should not think of the slime layer as a bulky mass. Where velocities are good, the submerged pipe wall usually appears clean, but the slime layer always gives the surface a characteristically slippery feel unless it is cleaned off by rubbing. It is significant that the highest oxygen flux to the pipe wall, 1.4 grams/m^2 hr, was in line 18J at a velocity of 7.1 fps (2.2 m/sec). The finding of a highly reactive slime layer where the stream is so swift is in a sense an extension of the finding, previously reported (3), that a velocity of 4 fps did not diminish sulfide production in a force main. Let no one suppose that high velocity will strip a pipe wall free of biologically active material, unless it be at some velocity greater than 7 fps, or unless much abrasive material is present.

Conditions of Deficient Oxygen Supply

Three of the reaches tested (48D, 27D, and 27J) had very low oxygen concentrations at the downstream ends. It was calculated in all of these cases that the oxygen concentrations had come close to the steady state values, where dO_2/dt contributed little or nothing to the balance. The rates of oxygen consumption equaled the rates of supply.

By using Eq. 7 to predict slime usage and Fig. 3, together with pot tests, to estimate R_s, predictions may be made of the steady state concentrations. Seventeen tests in the three lines mentioned showed oxygen concentrations ranging from 0.03 to 0.27 mg/l. Predicted concentrations were in the same range, but the standard error of the predictions for individual runs was 0.09 mg/l. In view of the errors of measurement and the fact that Fig. 3 represents only an average trend, closer agreement would not be expected; it is sufficient that the observed range is rationally accounted for.

Deductions

The significance of these findings for the problem of septicity in sewers will be treated elsewhere.

Wherever the slime layer acts as an efficient oxygen sink, and with normal surface aeration providing the only source of oxygen, concentrations will drop to low levels. The transfer coefficient to the slime layer is in general about 2½ times as great as the coefficient for absorption of oxygen from the air. If no oxygen were used in a stream of sewage, but the slime layer were active, and if the stream were broad and shallow, so that the surface area was practically equal to the slime area, the steady-state oxygen concentration would be 1/3.5 or 28% of saturation. In a half-filled sewer it would be 20% of saturation. In a very small sewer, with swift flow, oxygen consumption in the stream may be small compared to the through flux, and these concentrations might be approached, but the

actual consumption in streams more than a few inches deep in a warm climate, or more than 2 or 3 feet deep where sewage temperatures are low, will establish steady-state concentrations near zero.

In line 1B1 (Table 2) the oxygen concentration averaged 1.95 mg/l, but even with a stream velocity of 7.1 fps it was losing oxygen at a rate of 8.5 mg/l-hr; it would reach a steady-state concentration of 1.2 mg/l. Seven of the ten sets of data in Table 2 forecast steady-state concentrations less than 0.5 mg/l.

Oxygen concentrations higher than the steady state levels calculated from the prevailing flow conditions and observed K_2 are common, since surface aeration in uniform-flow reaches is not the only source. Junctions and other points of turbulence make significant contributions. There are also many sewers in which the slime layer is not 100% effective as an oxygen sink.

Where the oxygen supply is adequate, oxidation of the sewage in transit may be a significant factor. The South Slope Trunk of the Los Angeles County Sanitation Districts (Fig. 6) is illustrative. The oxygen supply is augmented by turbulence from steep tributary lines, so that the stream is fairly well aerated along much of its length. Estimates made on the basis of the data for January in Fig. 5 indicate that the total flow reacts with about 67 mg/l of oxygen by the time it reaches the pump station. In September, more oxygen is used by the stream, but use by the slimes is less, the total being about 85 mg/l. Another tributary flow enters just ahead of the Whittier Point pump station, and then the total flow is aerated by compressed air in the force main, consuming 16 mg/l of oxygen under average conditions. The oxygen absorbed in the force main partially compensates for the flow added near the pump station. Thus it appears that about 60 to 75 mg/l of oxygen, depending upon the season, react with the sewage from the time it enters the trunk until it discharges from the force main. This is under daytime flow conditions. The effect would be somewhat greater for the 24-hour average flow.

Six samples of sewage taken at the end of the force main in March and May showed BOD concentrations averaging 142 mg/l. An average BOD determined on a set of samples from the tributary flows, taken at similar hours and seasonal conditions, showed 195 mg/l. It appears well established that the sewage did receive a substantial degree of biological oxidation in the trunk and force main. Furthermore, some oxidation undoubtedly occurs in the tributary lines, adding to the total in-sewer BOD reduction that occurs in this instance. The trend toward regional sewerage systems means that an increasing number of large, long trunks will be built. Detention times will be long enough in some of these trunks to permit major BOD reductions, but this does not occur in such trunks because of oxygen starvation. If oxygen were supplied in sufficient amount, substantial reduction could be made in the levels reaching downstream treatment plants.

Summary:

1. Oxygen reaction rates in sewages vary widely, being low near the origin of the sewage, increasing for several hours if aerobic conditions prevail, up to rates as high as 30 mg/l-hr, then declining. This behavior was demonstrated both in laboratory experiments and in sewers.
2. By using a predictive equation for reaeration and measuring the rates of use of oxygen by the sewage, calculations were made of the rate of oxygen utilization by the slime layer on the pipe wall. It was hypothesized that in the sewers tested the slime layer acted as an efficient sink for the oxygen reaching it. The

results were consistent with this hypothesis. A predictive equation for the R_s, the rate of loss of oxygen to the slime layer, is

$$R_s = \frac{S\,k(O_s)\,(vr)^n}{f} \qquad (8)$$

3. In well oxidised streams of low oxygen demand, the slime layer may not be completely effective as an oxygen sink.
4. A substantial amount of biological oxidation occurs in some sewers, particularly where the oxygen supply is augmented by the turbulence produced at junctions and other structures. By designing sewers in ways to maximise aeration of the stream, or by supplying oxygen in other ways, sufficient biological oxidation may be induced to effect substantial reductions in the BOD loads on treatment plants.

SYMBOLS

C_s	general constant for the equation for transfer of oxygen to the slime layer, m/hr.
C_f	general constant for the predictive aeration equation, m/hr.
d_m	mean hydraulic depth, cross section area of stream divided by surface width, m or ft.
$(D)_1$, $(D)_2$	oxygen deficits at beginning and end of a test reach, mg/l.
e	oxygen transfer coefficient from water to a slime layer acting as an effective oxygen sink, m/hr.
F	Froude number, $\sqrt{d_m g}$.
k	oxygen exchange coefficient, m/hr.
(O_s)	oxygen concentration, grams per cubic meter (mg/l).
Q	flow, m^3/sec or cfs.
r	hydraulic radius, m or ft.
R_f	rate of supply of oxygen to a stream, g/m^3 hr.
R_r	rate of reaction of oxygen with impurities in the water, g/m^3 hr.
R'_r	rate of reaction of oxygen under the condition that there is no retardation due to a suboptimal concentration, g/m^3 hr.
R_s	rate of loss of oxygen from stream to slime layer, g/m^3 hr.
S	slope of the energy line of the stream.
T	temperature, °C.
t	time, seconds or hours.
v	velocity of flow, m/sec or ft/sec.
γ	ratio of exchange coefficient for oxygen absorption by a stream at temperature T to the coefficient at 20°C, other conditions being the same.
ϕ_s	flux of oxygen from the stream to the slime layer, g/m^3 hr.
ϕ_f	flux of oxygen from the atmosphere to the stream, g/m^3 hr.

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ACKNOWLEDGEMENTS

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Appreciation is expressed to Charles Swanson and Jerry Stern of the Environmental Protection Agency for their advice in the planning and execution of both projects.

APPENDIX II

CALCULATION OF EQUILIBRIUM CONDITIONS WHEN OXYGEN IS MIXED WITH SEWAGE

Assume a situation in which sewage has a dissolved nitrogen concentration in equilibrium with the atmosphere and no dissolved oxygen. If oxygen is mixed with this sewage, oxygen will dissolve and nitrogen will enter the gas phase. The problem is to calculate the equilibrium condition. It is to be assumed that the sewage is not retained in contact with the gas phase for a long enough time to allow any substantial biological consumption of the oxygen.

Symbols

P = pressure in the gas phase, atmospheres

T = temperature, °C

d_o = density of dry oxygen at P, T

d_n = density of dry nitrogen at P, T

H_o = vapor pressure of oxygen, atm., over a solution of 1 mg/l O_2 at T

H_n = vapor pressure of nitrogen, atm., over a solution of 1 mg/l of N_2 at T

V = volume of gas phase associated with one liter of water

V_o = partial volume of O_2 in gas phase associated with one liter of water

V_n = partial volume of N_2 in gas phase associated with one liter of water

P_w = vapor pressure of water, atm.

a' = initial oxygen concentration, mg/l

a = oxygen dissolved, mg/l

b = nitrogen leaving the water, mg/l

A = amount of oxygen supplied, mg/l of water

Solution

1. $P = P_o + P_n + P_w \qquad P_o + P_n = P - P_w$

2. $P_o = aH_o$

3. $P_n = 0.79(1 - P_w) = bH_n$

(The partial pressure of nitrogen in dry air at one atmosphere pressure is 0.79 atm., but in contact with water it is diluted by water vapor, hence the factor $1 - P_w$.)

$$P = aH_o + 0.79(1 - P_w) = bH_n + P_w$$

$$b = \frac{aH_o + 0.79(1 - P_w) - P + P_w}{H_n}$$

4. $$b = \frac{aH_o + 0.79 + 0.21P_w - P}{H_n}$$

The partial pressures of the gases are proportional to the partial volumes. Since $\frac{P}{P_o + P_n} = \frac{P}{P - P_w}$ it follows that

5. $$\frac{V}{V_o + V_n} = \frac{P}{P_o + P_n} = \frac{P}{P - P_w}$$

$$V = (V_o + V_n)\left(\frac{P}{P - P_w}\right)$$

6. $$V_o = \frac{h}{d_o} = \frac{g}{d_o}$$

7. $$V_n = \frac{h}{d_n}$$

8. $$V = \left(\frac{A}{d_o} - \frac{a}{d_o} + \frac{b}{d_n}\right)\left(\frac{F}{P - P_w}\right)$$

Since $\frac{P}{P^u} = \frac{V}{V^u}$ it follows that

9. $$\frac{V}{V^u} = \frac{\frac{A}{d_u} - \frac{a}{d_u}}{\left(\frac{A}{d_o} - \frac{a}{d_o} + \frac{b}{d_n}\right)\left(\frac{V}{V - P_w}\right)}$$

$$P_o = \frac{(P - P_w)\left(\frac{A}{d_u} - \frac{a}{d_u}\right)}{\frac{A}{d_o} - \frac{a}{d_o} + \frac{b}{d_n}}$$

$$P_o = \frac{(P - P_w)(A - a)}{A - a + \frac{bd_u}{d_n}}$$

Substituting all_o for P_o and equation 4 for b:

10. $$all_o = \frac{(P - P_w)(A - a)}{A - a + \frac{d_o}{d_n H_n}(all_o + 0.79 + 0.21P_w - P)}$$

$$Aall_o - a^2 H_o + \frac{ad_o H_o}{d_n H_n}(all_o + 0.79 + 0.21P_w - P) = (P - P_w)(A - a)$$

$$A(all_o - P + P_w) = a^2 H_o - \frac{ad_o H_o}{d_n H_n}(all_o + 0.79 + 0.21P_w - P) - a(P - P_w)$$

11. $$A = \frac{a(P - P_w) - a^2 H_o - \frac{ad_o H_o}{d_n H_n}(P - 0.79 - 0.21P_w - all_o)}{P - P_w - all_o}$$

$$12.\quad A = \frac{a(P - P_w - aH_o(1 - \frac{d_o H_o}{d_n H_n}) - \frac{d_o H_o}{d_n H_n}(P - 0.79 - 0.21P_w)]}{P - P_w - aH_o}$$

By inserting values of d_o; d_n; H_o; H_n and P_w, which are functions of temperature, and any chosen value for P, a simple equation is obtained showing how much oxygen must be applied to attain a desired oxygen concentration under equilibrium conditions. The table on the next page shows values of the temperature-dependent quantities used in the equation.

At low application rates, where all of the oxygen dissolves, the equation gives fictitious results, A being less than a. The equation that prevails in this range is A = a. As a approaches the solubility of pure oxygen in nitrogen-free water, A goes to infinity. At higher values of a, the equation gives fictitious negative values.

The proportion of oxygen in the residual oxygen-nitrogen mixture is

$$\frac{(A - a)H_n}{aH_o + 0.79 + 0.21P_w - P + (A - a)H_n}$$

The percentage efficiency of utilisation of oxygen is 100 a/A.

For the case that the water already contains dissolved oxygen at concentration a', the equation comparable to equation 11 is:

$$13.\quad A = \frac{a(P - P_w) - a(a + a')H_o - (a + a')\frac{d_o H_o}{d_n H_n}(P - 0.79 - 0.21P_w - aH_o - a'H_o)}{P - P_w - (a + a')H_o}$$

Figure 10 shows the results of calculations of the a vs. A relationship at 1 atmosphere pressure, 1.3 atmospheres (about 10 feet of water above atmospheric), and 1.6 atmospheres (20 feet of water).

COEFFICIENTS FOR EQUATION

FOR THE

OXYGEN EQUILIBRIUM CALCULATION

Temperature °C	H_o, atm. per mg/l	$\frac{d_o H_o}{d_n H_n}$	P_w, atm.
10	0.01861	0.4932	0.0168
15	0.02047	0.4993	0.0231
20	0.02251	0.5075	0.0313
25	0.02464	0.5147	0.0419
30	0.02670	0.5218	0.0555